PROCEEDINGS OF THE ELEVENTH

13 & 14 March 2020

INTERNATIONAL CIVIL ENGINEERING CONFERENCE

2020 ICEC

Integrating Innovation & Sustainability in Civil Engineering

Organized by

The Institution of Engineers Pakistan Karachi Centre

NED University of Engineering & Technology

in collaboration with

The Asian Civil Engineering Coordinating Council

Federation of Engineering Institutions of South & Central Asia (FEISCA)

Federation of Engineering Institutions of Islamic Countries (FEIIC)

Association of Consulting Engineers (Pakistan)

Sir Syed University of Engineering & Technology Karachi

Balochistan University of Engineering & Technology Khuzdar

Balochistan University of Information Technology Engineering & Management Sciences, Quetta

Mehran University of Engineering & Technology, Jamshoro

Quaid-e-Awam University of Engineering, Science & Technology, Nawabshah

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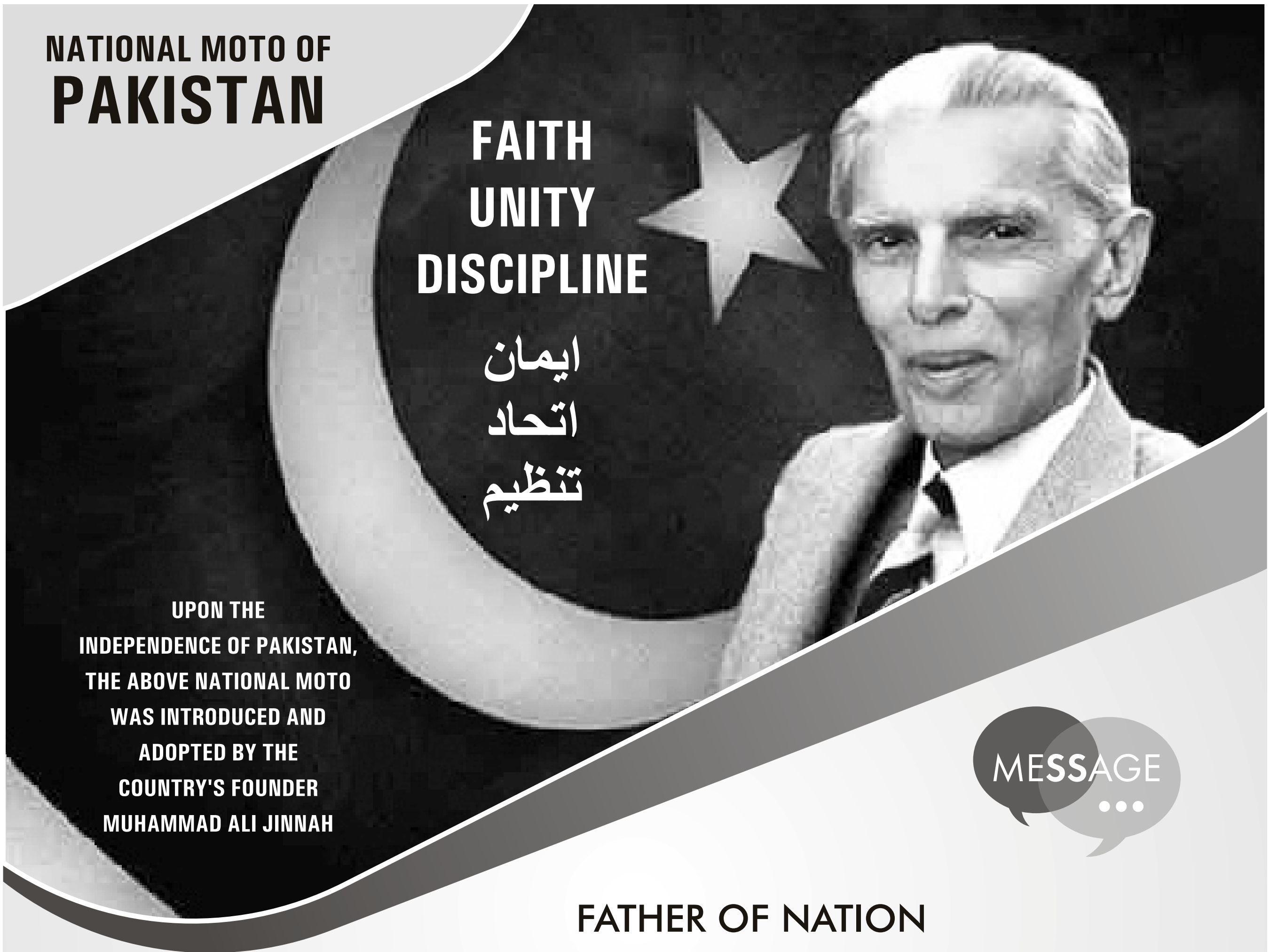

MESSAGE

FATHER OF NATION

MUHAMMAD ALI JINNAH

"If Pakistan is to take its proper place among the progressive nations of the world, it will have to take up a good deal of leeway in the realm of scientific and technical education which is so necessary for the proper development of the country and the utilization of its resources. The establishment of institution like the Institute of Engineers will greatly stimulate technical research and help in disseminating available information.

The Institute of Engineers will not only benefit the engineers themselves by improving their technical knowledge but also bring lasting benefits to public services which they are called upon to perform.

I wish the Institute every success"

QUAID-E-AZAM'S message to the first inaugural meeting of the Institute of Engineers Pakistan on 20th June 1948.

MESSAGE

Imran Ismail
Governor Sindh

"It is a matter of great pleasure to know that The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology are organizing the **11th International Civil Engineering Conference (ICEC-2020) on 13 & 14 March, 2020, on the theme "Integrating Innovation & Sustainability in Civil Engineering".** This Conference is a laudable platform that takes on board all major universities from across the country and aims to put our country on the path to scientific progress.

I am glad to learn that the Institution of Engineers Pakistan was established in 1948 with the blessings of the Father of Nation, Quaid-e-Azam Mohammad Ali Jinnah, who in his message to the first inaugural meeting of the Institution stressed that "If Pakistan is to take its proper place among the progressive nations of the world, it will have to take up a good deal of leeway in the realm of scientific and technical education which is so necessary for the proper development of the country and the utilization of its resources ".

It is encouraging to know that the principal aim of the 11th International Civil Engineering Conference will be to bring together the latest research and development in various fields of science and technology such as Civil engineering, Structural Engineering, Construction Engineering and Management, Earthquake Engineering, Transportation Engineering and Geo Technical Engineering and will prove beneficial to all participants.

I pray for the success of the 11th International Civil Engineering Conference and hope that similar events will be held in other disciplines of engineering as well.

MESSAGE

Mohammad Sarwar
Governor Punjab

"It gives me great pleasure to know that the Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology are organizing the 11th International Civil Engineering Conference (ICEC-2020) on 13th & 14th March, 2020 at Karachi in collaboration with Federation of Engineering Institutions of Islamic Countries (FEIIC), Federation of Engineering Institutions of South & Central Asia (FEISCA), and engineering and technology universities / institutions of Pakistan on the theme: **"Integrating Innovation & Sustainability in Civil Engineering".**

The 11th ICEC-2020 conference will provide an ideal nurturing platform for students, academicians, researchers and representatives from industry to mingle with each other in the most cordial environment for fruitful exchange of information, research ideas and promotion of collaborative research and development of culture for finding timely solutions to the current problems. The conference will feature technical paper presentations, final year project symposium, panel discussions and keynote speeches by eminent national and international scientists and leading industry personnel.

I sincerely hope that this year ICEC-2020 will forge strong ties between academics and industry to not only allow for finding indigenous solutions to our national problems but also help Pakistan to reap the benefits of a knowledge-based economy. I would like to appreciate the initiative by the Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering and Technology to include all stakeholders in academia and industry by taking them on board as technical partners and co-organizers.

I wish this conference a complete success!

MESSAGE

Engr. Syed Murad Ali Shah
Chief Minister Sindh

" I feel honored to know that the Institution of Engineers Pakistan and NED University of Engineering & Technology are holding the 11th International Civil Engineering Conference on 13 & 14 March, 2020 in collaboration with The Asian Civil Engineering Coordinating Council, Federation of Engineering Institutions of Islamic Counties and Federation of Engineering Institutions of South & Central Asia. Karachi Centre of the Institution of Engineers Pakistan has organized ten such conferences in the past and this is the eleventh in the series. I wish all those attending the conference a happy and comfortable stay of two days during the conference.

Engineers have played significant role in the overall development of the Country. The Institution of Engineers Pakistan has helped the engineers in widening their engineering knowledge and techniques by holding various technical activities. Its services to the nation are exemplary. The Institution of Engineers Pakistan has also played vital role by deliberating over the pertinent issues and making appropriate recommendations to the government. The 11th ICEC-2020 is one of the feature events of these continuing development efforts of the Institution of Engineers Pakistan (IEP) and NED University of Engineering & Technology (NEDUET).

I am sure the 11th International Civil Engineering Conference being attended by engineers from all over Pakistan and from around the world will provide an excellent opportunity to the participants to benefit from the experiences of one another and to find solutions to our current problems. The knowledge transferred by this Conference will be helpful for the participants, in increasing their professional ability and find ways and means to tackle the national and International problems.

The role played by the Institution of Engineers Pakistan is commendable and I wish every success for the Institution.

MESSAGE

Sardar Usman Ahmad Buzdar
Chief Minister Punjab

"It gives me immense pleasure to know that the Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology are jointly organizing the 11th International Civil Engineering Conference (ICEC-2020) being held on 13th & 14th March, 2020 on the theme "Integrating Innovation & Sustainability in Civil Engineering" in collaboration with The Asian Civil Engineering Coordinating Council (ACECC) Federation of Engineering Institutions of Islamic Countries (FEIIC), Federation of Engineering Institutions of South & Central Asia (FEISCA), Association of Consulting Engineers Pakistan, Balochistan University of Information Technology, Engineering & Management Sciences (BUITEMS), Balochistan University of Engineering & Technology, Khuzdar, Mehran University of Engineering and Technology, Sir Syed University of Engineering & Technology, and Quaid-e-Awam University of Engineering, Sciences & Technology.

The goal of ICEC-2020 is to gather scholars from all over the world to present advances in the relevant fields and to foster an environment conducive to exchanging ideas and information. This conference will also provide an ideal environment to develop new challenges and meet experts.

The Institution of Engineers Pakistan has been providing an excellent platform for participants to understand more about the latest development in the field of engineering, exchange views and experience, as well as brain storming.

I congratulate The Institution of Engineers Pakistan and NED University of Engineering and Technology for arranging the meaningful and thoughtful conferences on mega issue.

I wish all the success for 11th International Conference on Civil Engineering.

MESSAGE

Jam Kamal Khan
Chief Minister Balochistan

“I am extremely honored to express my deep hearted feeling of happiness that The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology, Karachi are organizing the 11th International Civil Engineering Conference (ICEC-2020), on 13 & 14 March, 2020 at Karachi. The conference is being organized in collaboration with The Asian Civil Engineering Coordinating Council (ACECC), Federation of Engineering Institutions of Islamic Countries (FEIIC), Federation of Engineering Institutions of South & Central Asia (FEISCA), Association of Consulting Engineers Pakistan, Balochistan University of Information Technology, Engineering & Management Sciences (BUITEMS) Quetta, Balochistan University of Engineering & Technology, Khuzdar, Sir Syed University of Engineering & Technology, Karachi. Mehran University of Engineering & Technology, Jamshoro and Quaid-e-Awam University of Engineering Sciences & Technology, Nawabshsh. The theme of the conference is “Integrating Innovation & Sustainability in Civil Engineering”

It is also a matter of great satisfaction that renowned experts from within the country and abroad shall be presenting their papers on the theme Integrating Innovation & Sustainability in Civil Engineering. All the Engineers shall greatly benefit from the Conference

The efforts of Institution of Engineers Pakistan, Karachi Centre are commendable for dissemination of knowledge by holding National, International Engineering Conferences, Technical Seminars, Workshops and lectures for the benefit of engineering community.

I pray for the success of this 11th International Civil Engineering Conference.

MESSAGE

Dr. Saad Al Shahrani
President, Federation of Engineering Institutions of Islamic Countries (FEIIC)

“The Federation of Engineering Institutions of Islamic Countries (FEIIC) is an international non-profit professional organization, established in 1989, with the aim of fostering cooperation in engineering education, research and professional practice in the Islamic Countries. It comprises of 22 member countries and a number of corporate and institutional members from amongst academic and research institutions, consultants, contractors and national organizations.

FEIIC, in cooperation with its members, has organized many scientific and research conferences, seminars, and workshops in its member countries on various aspects of engineering and related issues, such as engineering education, accreditation of engineering qualifications, and affordable housing etc. We are committed to share and exchange the experiences and expertise of the member countries with each other in addressing the crucial challenges in engineering and technological fields and in adopting the emerging trends and new concepts in engineering education, research and development and their implementation.

This 11th International Civil Engineering Conference (ICEC-2020) on “Integrating Innovation & Sustainable in Civil Engineering” is one of such efforts by the Institution of Engineers Pakistan, an active member of FEIIC, which we hope will bring the researchers and practicing engineers together on a shared platform to share and exchange their expertise and experiences.

Finally, I would like to congratulate and commend the partners and Organizing Committee of the Conference for all their efforts, and wish all the participants a very successful and enriching experience at the Conference.

MESSAGE

FEISCA

Eng. Jayavilal Meegoda
President – FEISCA
Immediate Past President - IESL

“ The Institution of Engineers Pakistan is playing a vital role in the Development of the Nation since its inception within the periphery of its approved aims and objectives, mostly revolving around the promotion and advancement of the practice and application of principles of Engineering, through its nine Centers spread across Pakistan and four overseas Centers. Upholding its traditions, the 11th International Civil Engineering Conference is being hosted by IEP-Karachi Centre this year. The Conference shall explore the latest technological development in the field of Civil Engineering and would broaden the vision of the participants.

On behalf of the Institution of Engineers Pakistan, Karachi Centre and the Organizing Committee, I would like to express my sincere appreciation for all participants, both from academia and industry, who played their role through contributions to the Conference and through their participation. In fact, all the members of the Technical Program Committee worked extremely hard to make this event happen. I have no doubt whatsoever that without their cooperation and their significant role and support, this event would not have been possible. Special thanks also goes to the keynote speakers, invited speakers, and authors, for strongly supporting the Conference, while there are no words to thank the Chief Guest/Guest of Honor who have spared their valuable time for this important event.

Finally, I welcome each participant and hope that they will find the 11th International Civil Engineering Conference not only useful in many respects but also to be a good opportunity to meet people and connect positively through networking in available time slots.

MESSAGE

Engr. Prof. Dr. Sarosh H. Lodi
Vice Chancellor,
NED University of Engineering and Technology, Karachi

"It gives me an immense pleasure to welcome you to the 11th International Civil Engineering Conference (ICEC-2020) which is jointly organized by The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology in collaboration with International organizations and regional universities. This conference provides the platform to researchers, academics, engineers and experts, not only from Pakistan but also from different countries around the world to share their research in the field of Civil Engineering and explore possibilities for collaboration in various fields.

The theme of the Conference i.e. "Integrating Innovation and Sustainability in Civil Engineering" is of great significance. The theme is aligned with the fact that Civil Engineering profession has been evolving rapidly with recent technological innovations and integration with other disciplines of Engineering. However, we need to integrate these innovations with sustainability principles for a better built environment. Moreover, the technological innovations specifically in construction technology, use of artificial intelligence (AI), virtual reality (VR), 3D printing and other innovations in Civil Engineering profession are transforming traditional civil engineering world over. Therefore, it is important to discuss and deliberate such transformation to keep up with the developed world. Such efforts will help towards sustainable built environment and achievement of sustainable development goals (SDGs).

I am sure that 11th International Civil Engineering Conference (ICEC-2020) will provide an excellent opportunity to the participants to benefit from the experiences of one another.

I wish all participants a successful conference in the beautiful city of Karachi. The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology would feel immense pleasure to welcome you in future conferences as well.

MESSAGE

Engr. Ahmed Farooq Bazai (S.I)
Vice Chancellor
Balochistan University of IT, Engineering and Management Sciences (BUITEMS)

“ It is my pleasure to be part of the **11th International Civil Engineering Conference (ICEC-2020)** I congratulate the Institution of Engineers Pakistan, Karachi Center and NED University of Engineering & Technology for organizing the conference series. The conference has successfully contributed to promoting progressive academic exploration and collaborations among the key stakeholders including the institutions of higher learning, industry and the government. With a prodigious demand for technological solutions, there is an everlasting need for innovative measures to cope with the strains of the forth industrial revolution. In this regard, **“Integrating Innovation & Sustainability in Civil Engineering ”** is worthy of the efforts; providing a platform to foster advancements within the domains of civil engineering, promoting inter-disciplinary cross-cutting technological breakthroughs in a variety of fields.

BUITEMS is especially focused on equipping the youth of today to meet future technological challenges by furnishing them with ample research and collaboration opportunities. As a collaborator of the ICEC-2020, we are certain of the opportunities provided under the theme. We hope that our partnership in this conference would go a long way and it will prove to be prolific to our faculty and students.

I sincerely commend the efforts of the entire organizing team in making this conference possible. I acknowledge the collaboration of the Asian Civil Engineering Coordinating Council (AECC), Federation of Engineering Institutions of Islamic Countries (FEIIC), Federation of Engineering Institutions of South & Central Asia (FEISA), Association of Consulting Engineers Pakistan, Mehran University of Engineering and Technology, Sir Syed University of Engineering & Technology, Quaid-e-Awam University of Engineering, Sciences & Technology, Balochistan University of Engineering & Technology, Khuzdar and Balochistan University of IT, Engineering and Management Sciences (BUITEMS), Quetta in jointly making this conference a success.

On behalf of BUITEMS, I would like to extend my warm wishes to all the delegates and participants.

MESSAGE

Prof. Dr. Ehsanullah Khan Kakar
Vice-Chancellor
Balochistan UET ,Khuzdar

“ It is a matter of great pleasure and honor for me and Balochistan University of Engineering & Technology, Khuzdar to be a part of 11th International Civil Engineering Conference-2020, which is going to be held at Karachi, Pakistan.

The 11th ICEC-2020, will focus on “Integrating Innovation & Sustainability in Civil Engineering'. The aim of the conference is to provide a premier platform for civil engineers and researchers from Pakistan and abroad to present their research experiences and ideas in the domain of Civil Engineering.

The Institution of Engineers Pakistan and NED University of Engineering & Technology in collaboration with various other universities organizes national and international conferences at regular basis. These academic events make the industry-academia linkages stronger and provide a better platform for future developments.

The conference objective is focused on the new and challenging problems related to Civil Engineering in ICEC-2020, the experts around the globe will share the new trends, solutions, in their respective areas of research which will benefit the conference participants at large.

Best wishes for the successful organization of the event and comfortable stay as well as for the planned souvenir on the occasion

MESSAGE

Engr. Dr. Javed Younas Uppal
President,
The Institution of Engineers, Pakistan

"I am extremely honored to express my deep hearted feeling of happiness that the Institution of Engineers Pakistan Karachi Centre and NED University of Engineering & Technology are holding the 11th International Civil Engineering Conference (11th ICEC-2020) in collaboration with The Asian Civil Engineering Coordinating Council, (ACECC), Federation of Engineering Institutions of Islamic Countries (FEIIC) Federation of Engineering Institutions of South & Central Asia (FEISCA), Association of Consulting Engineers Pakistan (ACEP),Baluchistan University of Engineering & Technology, Khuzdar (BUET), Balochistan University of Information Technology, Engineering and Management Sciences (BUITEMS) Sir Syed University of Engineering & Technology (SSUET) , Mehran University of Engineering & Technology, Jamshoro (MUET) and Quaid-e-Awam University of Engineering, Sciences & Technology, Nawabshah (QUEST) scheduled to be held on **Friday 13th & Saturday 14th March, 2020".**

It is an honor for the Institution of Engineers Pakistan that The Asian Civil Engineering Coordinating Council, (ACECC), is also holding its Executive Committee Meeting, Planning Committee Meeting (PCM), Technical Coordination Committee Meeting (TCCM), **Seminar on Sustainable Infrastructure** (TC-14) and **Seminar on Retrofitting and Strengthening of Existing Infrastructure** (TC-22) and an **International Workshop on Encouraging Youth and Young Professional's Involvement Towards Achievement of SDG's.**

It is also a matter of great satisfaction that renowned experts from within the country and from abroad shall be presenting their papers on the topic **"Integrating Innovation & Sustainability in Civil Engineering"** All the Engineers attending the conference shall benefit from the experience in their respective fields.

The Institution of Engineers Pakistan, Karachi Centre is working hard for dissemination of knowledge by holding National, International Engineering Conferences, Technical Seminars, Workshops and Lectures for the benefit of engineering profession and development of the Country.

The Chairman, IEP Karachi Centre Engr. Sohail Bashir, Vice-Chairman (Civil and allied) and Secretary, IEP Karachi Centre Engr. M. Farooq Arbi, Vice-Chancellor, NED University of Engineering and Technology Engr. Prof. Dr. Sarosh Hashmat Lodi, Members of the Organizing Committee and Central/Local Council Members of Karachi Centre deserve appreciation for organizing the 11th ICEC- 2020 for the benefit of engineering community.

I pray for the success of this 11th International Civil Engineering Conference.

MESSAGE

Engr. Amir Zamir Ahmed Khan
Secretary General
The Institution of Engineers, Pakistan

"It is a matter of great pleasure that the Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology are holding 11th International Civil Engineering Conference (11th ICEC-2020) in collaboration with The Asian Civil Engineering Coordinating Council, (ACECC), Federation of Engineering Institutions of Islamic Countries (FEIIC) Federation of Engineering Institutions of South & Central Asia (FEISCA), Association of Consulting Engineers Pakistan (ACEP),Baluchistan University of Engineering & Technology, Khuzdar (BUET), Balochistan University of Information Technology, Engineering and Management Sciences (BUITEMS) Sir Syed University of Engineering & Technology (SSUET) , Mehran University of Engineering & Technology, Jamshoro (MUET) and Quaid-e-Awam University of Engineering, Sciences & Technology, Nawabshah (QUEST) scheduled to be held on **Friday 13th & Saturday 14th March, 2020"**

The Institution is the premier body of qualified engineers in Pakistan and has made significant contributions to the development of the country. The role played by the Institution in spreading modern skills and technology is highly commendable. Recent advancements in Science and Technology have placed enormous power at the disposal of man which must be harnessed for the welfare of humanity. Pakistan possesses vast natural resources and it is the duty of our engineers to utilize these resources for the welfare of the society and eradication of disease, ignorance, poverty and hunger.

As Secretary General of the Institution of Engineers Pakistan I appreciate the efforts of Engr. Sohail Bashir, Chairman, IEP Karachi Centre for his effort in holding the Executive Committee Meeting of The Asian Civil Engineering Coordinating Council, (ACECC), Planning Committee Meeting (PCM), Technical Coordination Committee Meeting (TCCM), **Seminar on Sustainable Infrastructure** (TC-14) and **Seminar on Retrofitting and Strengthening of Existing Infrastructure** (TC-22) and an **International Workshop on Encouraging Youth and Young Professional's Involvement Towards Achievement of SDG's** alongwith the 11th ICEC-2020.

I am sure that 11th International Civil Engineering Conference being attended by engineers from all the provinces of Pakistan and also from abroad will provide an excellent opportunity to the participants to benefit from the experiences of one another in the light of Theme of the Conference **"Integrating Innovation & Sustainability in Civil Engineering".**

I wish the Institution of Engineers Pakistan Karachi Centre and Participants of the Conference all the success.

MESSAGE

Engr. Sohail Bashir, FIE, PE

Chairman, The Institution of Engineers Pakistan, Karachi Centre
Member, Executive Committee - FEIIC, FEISCA & ACECC

“ The Institution of Engineers Pakistan (IEP) is playing a vital role in the development of Pakistan since its inception within the frame work of its aims & objectives which revolves around the promotion of technology, advancement of the engineering practice, application of principles of science in engineering and dissemination of technical knowledge. Upholding its tradition continuously for the last ten years, this year also the 11th International Civil Engineering Conference is being hosted by the IEP Karachi Centre with more zeal and enthusiasm. The theme for this year conference is **“Integrating Innovation & Sustainability in Civil Engineering”**

The conference shall dwell on the latest technological development in the field of Civil Engineering and allied engineering disciplines which would not only broaden the vision of participants but shall led them to the frontiers of the existing knowledge and the way forward. Indeed to hold such International gathering, in the present security scenario and global Corona Virus threats was not only a challenge but was also an uphill task for which IEP Karachi Centre, NEDUET and all collaborating Institutions deserves all commendation. The collaborative role of Department of Civil Engineering NEDUET, BUITEMS, BUET-Khuzdar, SSUET, QUEST- Nawabshah and MUET-Jamshoro deserves special commendation.

On behalf of The Institution of Engineers Pakistan, Karachi Centre and the Organizing Committee of ICEC-2020, I would like to express my sincere appreciation for active participation, both from academia and industry. Indeed, all the members of Advisory Board, IEP Headquarters Committee, Management Committee, and Technical Review Committee worked extremely hard to make this event happen. I have no doubt whatsoever that without their cooperation, support and active participatory role, this event would not have been possible for which I record my appreciation for all of them. Special thanks to the Conference Key Note Speaker of Inaugural session, Dr. Kenichi Horikoshi, Secretary General, Asian Civil Engineering Coordinating Council who despite traveling advisory/travel restrictions, on our special request travelled from Japan to attend this Conference & ACECC meetings. I am also thankful to Engr. M. Najeeb Haroon, Member National Assembly of Pakistan our Guest of Honor in Inaugural Session for sparing his time and to Syed Murtaza Azhar Rizvi, Vice-President, Sindh Engro Coal Mining Company for his Key Note address on Thar Coal in the closing session of this conference. My profound thanks to Engr. Prof. Dr. Syed Tauha Hussain Ali, Pro Vice-Chancellor, MUET, our Guest of Honor in the closing session for coming from Hyderabad to grace this Conference by his presence Thanks to all invited speakers from industry, authors and sponsors for strongly supporting the conference. My sincere gratitude are to Engr. Prof. Dr. Sarosh Hashmat Lodi, Vice Chancellor, NEDUET & Convener, ICEC-2020 for his guidance & help in organizing ICEC-2020.

I would like to take this opportunity to place on record my sincere appreciation for Engr. Prof. Dr. Abdul Jabbar Sangi, Co-Convener, ICEC-2020, Engr. Dr. Shamsoon Fareed, Secretary, ICEC-2020, Engr. Shoaib Ahmed Co-Secretary, ICEC-2020, Engr. Faiza Saeed and the student volunteers of NEDUET for their hard work for ICEC-2020.

Finally, I would like to welcome each one of the participant and hope that they will find ICEC-2020 not only useful in enhancing their technical knowledge but also to be forum to meet many highly respected engineers under one roof for effective interaction in future.

MESSAGE

Engr. Prof. Dr. Muhammad Tufail
Pro Vice Chancellor
NED University of Engineering & Technology

“ It indeed is my proud privilege to be part of the 11th International Civil Engineering Conference (ICEC 2020) which is jointly organized by The Institution of Engineers Pakistan, Karachi Centre and NED University of Engineering & Technology in collaboration with International organizations and regional universities.

It is heartening to know that this time more than 60 research articles shall be presented which will be focusing on the core issues in line with the conference theme which is "Integrating Innovation & Sustainability in Civil Engineering". I am sure this conference will provide the platform to researchers, academics, engineers and experts, not only from Pakistan but also from different countries around the world to share their research in the field of Civil Engineering and explore possibilities for collaboration in various fields.

The theme of the conference focuses on integrating innovation and sustainability in Civil Engineering. For a developing country like Pakistan, it is of paramount importance that engineers and policy makers start working towards implementing and achieving sustainable development goals (SDGs) for the development of the country. I am confident that the participants of the conference will greatly benefit from the research work to be presented.

I wish all participants and organizers a successful conference.

MESSAGE

Engr. Prof. Dr. Asad-ur-Rehman Khan
Dean
Faculty of Civil & Petroleum Engineering
NED University of Engineering & Technology

"I am pleased to be part of the 11th International Civil Engineering Conference being organized by the Institution of Engineers Pakistan and Department of the Civil Engineering of the NED University of Engineering & Technology on the 13th & 14th March 2020. The conference is being organized in collaboration with The Asian Civil Engineering Coordinating Council, Federation of Engineering Institutions of Islamic Counties, Federation of Engineering Institutions of South & Central Asia and the Association of Consulting Engineers of Pakistan.

Engineers have an important role in the development of any country and platforms like this conference provide an excellent opportunity to keep themselves abreast of the latest advancements and innovations in the Engineering field.

The 11th ICEC is one of the feature events of these continuing development efforts of the Institution of Engineers Pakistan (IEP) and NED University of Engineering & Technology (NED) and this time around 60 technical papers shall be presented which will be focusing on the core issues in the line of the conference theme which is "Integrating Innovation & Sustainability in Civil Engineering".

I am sure that the 11th International Civil Engineering Conference will provide a platform to the participants to benefit from the experiences of one another and to find solutions to our current problems. The knowledge transferred by this Conference will be helpful for the participants, in increasing their professional abilities and find ways and means to tackle the national and International problems.

I wish all those attending the conference a happy and comfortable stay of two days during the conference.

MESSAGE

Engr. Prof. Dr. Rizwan Ul Haq Farooqui
Chairman
Department Civil Engineering
NED University of Engineering & Technology

"It gives me immense pleasure to share my views on the 11th International Civil Engineering Conference (ICEC-2020) jointly organized by Department of Civil Engineering, NED University of Engineering & Technology and The Institution of Engineers Pakistan Karachi Centre. As Chair of the Department of Civil Engineering at NED University, it is very heartening to see that the series of Conferences have come a long way over the past years and we are already into the eleventh chapter of the series. We have been getting a wonderful response in the form of scholarly contributions from both national as well as international arena. For this particular Conference, over 60 articles under 10 technical streams within Civil Engineering and 15 parallel sessions are being presented over two days in addition to Key Notes and Invited Talks from industry, which in itself is an indicator of success of the event. In a country like Pakistan where research and development are not, unfortunately, taken as national priorities, such a healthy level of contribution from both academics and industry are pivotal for national success. The theme of the Conference i.e. "**Integrating Innovation and Sustainability in Civil Engineering**" has been set around the key idea that with the 4th Industrial Revolution and the drive towards sustainability, civil engineers have to play a pivotal role towards developing and advancing technology-driven sustainable built environment, particularly in the developing side of the world in order to address the indigenous issues. I am very hopeful that the papers presented in the conference would provide useful recommendations towards sustainable societal development via civil engineering innovation.

I would personally like to acknowledge the endless efforts extended from IEP, NED committee members, partner Universities, authors, invited speakers from the industry, keynote speakers and volunteers for making this event a success. I sincerely hope that the Conference would pave way for further strengthening the academia-industry linkage and coming up with sustainable, innovative and collaborative solutions to our indigenous issues in the built environment.

MESSAGE

Engr. M. Farooq Arbi, FIE, PE
Secretary,
The Institution of Engineers Pakistan, Karachi Centre

"I am honored to warmly welcome all of you to the 11th International Civil Engineering Conference (ICEC- 2020) being jointly organized by The Institution of Engineers Pakistan Karachi Centre and NED University of Engineering & Technology on 13th & 14th March, 2020 at IEP Karachi This time our collaboration include Federation of Engineering Institutions of Islamic Countries, Federation of Engineering Institutions of South & Central Asia, The Asian Civil Engineering Coordinating Council, Balochistan University of Information Technology Engineering & Management Sciences (BUITEMS), Mehran University of Engineering and Technology, Sir Syed University of Engineering & Technology, Quaid-e-Awam University of Engineering Sciences & Technology, Balochistan University of Engineering and Technology, Khuzdar, on the theme "Integrating Innovation & Sustainability in Civil Engineering".

The Institution of Engineers Pakistan is playing a vital role in the Development of the Nation since its inception within the periphery of its approved aims and objectives, mostly revolving around the promotion and advancement of the practice and application of principles of Engineering, through its nine Centres. Spread across Pakistan and four overseas Centres. Upholding its traditions, the 11th International Civil Engineering Conference is being hosted by IEP-Karachi Centre this year.

The theme of the Conference i.e "Integrating Innovation & Sustainability in Civil Engineering" is of great significance, The theme is aligned with the fact Civil Engineering profession has integrated technology form many other disciplines for enhancing its efficiency and productivity.Moreover, the technology specifically that of construction technology has also gone through prosperous evolution. Use of artificial intelligence (AI) virtual reality (VR), 3D printing and other technological advancements in Civil Engineering profession are transforming traditional civil engineering. Future of Civil Engineering profession lies in technology. Therefore, it is important to discuss and deliberate such transformation to keep up with the developed world.

I take this opportunity to express my appreciation to the joint efforts of Engr. Prof. Dr. Sarosh Hashmat Lodi, Vice-Chancellor & Convener, ICEC-2020, NEDUET and Engr. Sohail Bashir, Chairman, IEP Karachi Centre, for the success of this conference. Special thanks to Engr. Prof. Dr. Abdul Jabbar Sangi, Co-Convener, ICEC-2020, Engr. Dr. Shamsoon Fareed, Secretary ICEC-2020 and Engr. Shoaib Ahmed Co-Secretary, ICEC-2020, Engr. Faiza Saeed and my of ce staff for their untiring effort for ICEC-2020.

Finally, I welcome each participant and hope that they will nd the 11th International Civil Engineering Conference not only useful in many respects but also to be a good opportunity to meet people and connect positively through networking in available time slots.

MESSAGE

Engr. Dr. Abdul Jabbar Sangi
Co-Convener, ICEC-2020
NED University of Engineering and Technology

"It is my proud privilege to be the Co-Convener of the 11th International Civil Engineering Conference (ICEC-2020) jointly organized by The Institution of Engineers Pakistan Karachi Centre and NED University of Engineering & Technology in collaboration with The Asian Civil Engineering Coordinating Council, Federation of Engineering Institutions of Islamic Countries, Federation of Engineering Institutions of South & Central Asia and Association of Consulting Engineers of Pakistan on Friday 13th & Saturday 14th March, 2020 in Karachi. The event is organized in partnership with regional universities including Balochistan University of Engineering & Technology Khuzdar, Balochistan University of Information Technology, Engineering & Management Sciences (BUITEMS) Quetta, Sir Syed University of Engineering & Technology, Karachi, Mehran University of Engineering & Technology, Jamshoro and Quaid-e-Awam University of Engineering, Science and Technology, Nawabshah.
This year, the 11th International Civil Engineering Conference (ICEC-2020) is being held under the theme "**Integrating Innovation and Sustainability in Civil Engineering**". The idea is to bring together latest civil engineering knowledge, research and development efforts from scientific community, engineers and practitioners focusing on recent innovations incorporating notions of sustainability that can help in contributing towards establishing a better built environment and achieving sustainable development goals. The multiple challenges faced by developing countries related to the built-environment can only be solved by adopting innovative technological approach towards the development process based principles of sustainability.

I would like to thank IEP and NED committee members, partner Universities, authors, invited speakers from the industry and volunteers for their valuable contribution towards the event. I hope that this conference would strengthen further the meaningful interactions between industry, academia and scientific community, which will enable further research & development and help develop sustainable built-environment in the country.

11th ICEC-2020 ORGANIZING TEAMS

ADVISORY BOARD

Engr. Prof. Dr. Sarosh Hashmat Lodi
Vice Chancellor NEDUET & Convenor ICEC-2020

Engr. Sohail Bashir
Chairman, IEP Karachi Centre

Prof. Dr. Mohammad Aslam Uqaili
Vice Chancellor, Mehran UET

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PROGRAMME

DAY -1 INAUGURAL SESSION
at NED University Auditorium - Friday 13th March, 2020

03:00-3:20 pm	**Registration/Networking**
03:25 pm	**Guest to be seated**
03:30 pm	**Recitation from the Holy Quran**
03:35 pm	**National Anthem**
03:40 pm	Conference Briefing by **Engr. Prof. Dr. Abdul Jabbar Sangi** Co-Convener, ICEC-2020
03:45 pm	Welcome Address by **Engr. Sohail Bashir** Chairman, IEP, Karachi Centre
03:50 pm	Key Note Address on **"Recent Technology Development in Japanese Construction Industry"** by **Dr. Kenichi Horikoshi** Secretary General Asian Civil Engineering Coordinating Council, Vice-President, Japanese Geotechnical Society General Manager, Taisei Corporation, Japan
4:20 pm	Address by **Engr. Prof. Dr. Sarosh Hashmat Lodi** Vice-Chancellor, NEDUET & Convener, ICEC-2020
04:25 pm	**Address by Guest of Honor** **Engr. Najeeb Haroon,** Member National Assembly of Pakistan
04:30 pm	**Address by Chief Guest**
04:35 pm	**Presentation of Conference Memento**
04:40 pm	**Vote of Thanks by** **Engr. M.Farooq Arbi** Secretary, IEP, Karachi Centre
04:45 pm	Asr Prayers
05:00 pm- 6:30 pm	
Technical Session- 1	**"Structural Engineering"**
Technical Session-2	**"Water Resources Engineering"**
Technical Session-3	**"Construction Engineering"**
6:40 pm	**Maghrib Prayers**
7:00 pm - 8:00 pm	Socialising & Entertainment Session by Engineers
8:00 pm	**Conference Dinner**

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PROGRAMME

Day 2 - Saturday 14th March, 2020
at Convention Center 5th Floor, IEP Building, Shahrah-e-Faisal, Karachi

09:30 am 0 11:00 am

Technical Session-4	**"Water Resources Engineering"**
Technical Session-5	**"Structural Engineering"**
Technical Session-6	**"Materials Engineering"**
Technical Session-7	**"Construction Engineering"**
11:00- 11:15 pm	**Tea Break**

11:15-1:15 pm

Technical Session-8	**"Water Resource Engineering"**
Technical Session-9	**"Structural Engineering"**
Technical Session-10	**"Materials Engineering"**
Technical Session-11	**"Geotechnical Engineering"**
1.15pm-2:00 pm	**Zohar Prayers/Lunch**

02:00-4:00 pm

Technical Session-12	**"Structural Engineering"**
Technical Session-13	**"Materials Engineering"**
Technical Session-14	**"Structural Engineering"**
Technical Session-15	**"Infrastructure Engineering**

CLOSING SESSION

04:00-04:10 pm	**Guest to Seated/Networking**
04:10 pm	**Recitation from Holy Quran**
04:15 pm	Conferencing Highlights by **Engr. Prof.Dr. Rizwan ul Haque Farooqui** Chairman, Department of Civil Engineering, NEDUET
04:20 pm	Address by **Engr.Sohail Bashir**, Chairman, IEP, Karachi Centre
04:25 pm	Address by **Engr. Prof.Dr. Asad- ur Rehman Khan** Dean, Faculty of Civil & Petroleum Engineering, NEDUET
04:30 pm	Keynote Address by **Syed Murtaza Azhar Rizvi** Vice President, Sindh Engro Coal Mining Company (SECMC). on **"Thar Coal"**
05:00 pm	Address by **Engr. Prof. Dr. Sarosh Hashmat Lodi** Vice-Chancellor, NEDUET
05:05 pm	**Address by Guest of Honor** **Engr. Prof. Dr. Syed Tauha Hussain Ali** **Pro-Vice Chancellor** **Mehran University of Engg. & Tech.**
05:10 pm	**Address by Chief Guest**
05:15 pm	**Presentation of Chairman IEP** **Medal for Best Paper**
05:20 pm	Conference Recommendations by **Engr. Dr. Shamsoon Fareed** Secretary, ICEC-2020
05:25 pm	**Presentation of Conference Mementos**
05:30 pm	Vote of Thanks by **Engr. M. Farooq Arbi** Secretary, IEP, Karachi Centre
05:35 pm	Asr Prayers
05:50 pm	Hi-Tea

TECHNICAL PROGRAMME

ICEC 2020 Parallel Technical Sessions
Day 1- Friday, March 13, 2020
PARALLEL TECHNICAL SESSIONS
Time: 5:00-6:30 pm
Venue: NED University of Engineering & Technology, Karachi

Technical Session 1 Structural Engineering **Time**: 5:00-6:30 pm **Venue: Civil Video Conferencing Hall, NEDUET** **Session Chairs** Engr.Prof .Dr. S. F.A.Rafeeqi,IEP Engr. Askhar Dawar, IEP	**Technical Session 2** Water Resource Engineering **Time**: 5:00-6:30 pm **Venue: CIS Lecture Hall, NEDUET** **Session Chairs** Engr. Dr. Syed Imran Ahmed, NEDUET Engr. Javed Aziz Khan, IEP	**Technical Session 3** Construction Engineering **Time**: 5:00-6:30 pm **Venue: Mechanical AV Hall, NEDUET** **Session Chairs** Engr. Dr. Rizwan Farooqui, NEDUET Engr. Bushran Nadeem Mufti, IEP
Seismic Performance of Moment Resisting Reinforced Concrete Frames under Code Compatible Ground Motions *Naik Muhammad, Abd Ullah, Mahnoor Mirwani, Faiza Nadeem and Saeed Ullah Jan Mandokhail*	**Numerical Modelling of Three-Dimensional Flow Structure in Open Channel having Patches of Aquatic and Riparian Vegetation** *Muhammad Asif and Usman Ghani*	**Comparison of Green Building Rating Tools by Assessment of Residential Apartment** *Usman Hussain and Shujaa Safdar*
Fire Protection and Safe Evacuation Issues for Congested Areas in Pakistan *Muhammad Imran*	**Experimental Investigation on Different Geometrical Shapes of Dual Bridge Pier Scouring** *Akhtar Abbas and Usman Ghani*	**Resource Procurement and handling Issues Generating Waste in Construction Projects** *Shahzad Ali, Aftab Hameed Memon, Tauha Hussain Ali, Shabir Hussain Khahro, Nafees Ahmed Memon*
Condition Assessment of an Aging Bridge: A Case Study *Rashid Ahmed Khan, Aslam Faqeer Mohammad, Abdul Jabbar Sangi and Amir Nizam*	**A Ninth Century Earthquake induced Landslide Breaching and Downstream flood impact assesment on River Jhelum** *Muntaha Ateeq, Roger Bilham and Imran Ahmed*	**An ultimate Reward-based Blockchain deployment for road user safety** *Muhammad Mursaleen, M. Abdul Aziz ,Ijaz Nazir Mehrr*
Invited Talk by **Engr. Muhammad Najeeb Haroon**	**Invited Talk by** **Engr. Dr. Bashir Lakhani**	**Invited Talk by** **Engr. Balal Khawaja Advocate**

ICEC 2020 Parallel Technical Sessions
Day 2- Saturday, March 14, 2020
PARALLEL TECHNICAL SESSIONS
Time: 9:30am to 11:00 am
Venue: IEP Convention Center, 5th Floor, IEP Building, Shahrah-e-Faisal, Karachi

Technical Session 4 Water Resource Engineering **Time:** 9:30am to 11:00 am **Venue: Engr. Asfia Aleem Hall** **Session Chairs** Engr. Dr.M. Shafqat Ejaz, NEDUET Engr. Mukesh Kumar, IEP	**Technical Session 5** Structural Engineering **Time**: 9:30am to 11:00 am **Venue: IEP Convention Centre** **Session Chairs** Engr.Dr. Tehmina Ayub, NEDUET Engr. Umar Mashkoor Makhdumi,IEP	**Technical Session 6** Materials Engineering **Time:** 9:30am to 11:00 am **Venue: Dr. S.A. Hassan Hall** **Session Chairs** Engr.Dr. Aftab Memon, QUEST Engr. Shakeel Ahmed , BUT-Khuzdar	**Technical Session 7** Construction Engineering **Time:** 9:30am to 11:00 am **Venue: Seminar Hall** **Session Chairs** Engr.Dr. Nafees Memon, MUET Engr. Salhauddin, BUT-Khuzdar
Impact on Local Scouring Around Bridge Pier by Submerged Trapezoidal Broad Crested Weir *Muhammad Asadullah and Naeem Ejaz*	**Effects of Stirrup,s spacing on the confinement, ductility, strength and curvature of circular column using matlab cumbia code** *Syed Waqar Younas, Dr. Shahid Ullah, Tawqeer Alam and Bilal Siddiq*	**A review on observed flaws in bricks and their possible sustainable remedies for local construction** *Waleed Raza, Faiza Khalid and Majid Ali*	**Adoption of Building Information Modeling (BIM) in Construction Industry of Pakistan: Benefits and Barriers** *Muhammad Saleem Raza, Danish Kumar, Hassnain Nawab, Aqeel Murtaza, Shoail Farooq and Zaid Khan Pathan*
Investigating the Effects of Finite Length Rigid Emergent Vegetation on Inland Flood Attenuation *Kamran Tariq, Ghufran Ahmed Pasha, Norio Tanaka, Usman Ghani and Afzal Ahmed*	**Comparative study on seismic analysis and design of high-rise building using static and dynamic analysis by Etabs** *Abrar Ahmad, Mamoon Riaz, Tayyab Zafar and Kanwar Kashif Zulfiqar*	**A Review On Different Influential Factors And Techniques For Evaluating Bonding Between Asphalt Overlay And Concrete Pavement** *Muhammad Ali Tariq, Muhammad Usman Farooqi and Majid Ali*	**Schedule performance in dam construction projects of Sindh** *Sohail Ali Mallah, Nafees Ahmed Memon, Aftab Hameed Memon, Samiullah Sohu, Naeem Aziz Memon*
To Investigate the Breaching Phenomenon of a Fuse plug *Zain Ud Din, Ghufran Ahmed Pasha, Usman Ghani and Afzal Ahmed*	**Improvement In Impact Resistance Of GFRP Reinforced Concrete Wall Panels Using Jute Fibres** *Shehryar Ahmed and Majid Ali*	**Effect of Fly Ash and Polypropylene Fiber on Compressive Strength of Concrete at Normal as Well as Elevated Temperatures** *Ghulam Qanber and Faisal Shabbir*	**Use of BIM Tools In Highway Transportation System In Developed Countries - A Review** *Misbah Ur Rehman, M. Usman Farooqi and Majid Ali*
	Numerical investigation of GFRP Reinforced Non-Circular Concrete Column with Fibre-glass Grating Mesh (FGM) Ties *Muhammad Fawad Rashid, Afaq Ahmad and Mohamed Elchalakani*	**Utilization of Natural Fibers in Concrete for Improving Behavior of Non-Structural Elements – A Review** *Ahsan Abbas, Ali Ejaz and Majid Ali*	**Sustainable Construction in Balochistan: Issues and Challenges** *Shakeel Ahmed, Syed Abdullah Shah Hashmi, Salah Uddin, Nafees Ahmed Memon*
Invited Talk by **Mr. Mujib Baloch**	**Invited Talk by** **Engr. Muhammad Zahid Zia**	**Invited Talk by**	**Invited Talk by** **Engr. Ghulam Mujtaba Shaikh**

TECHNICAL PROGRAMME

ICEC 2020 Parallel Technical Sessions
Day 2- Saturday, March 14, 2020
PARALLEL TECHNICAL SESSIONS
Time: 11:15 to 1::15 pm
Venue: IEP Convention Center, 5th Floor, IEP Building, Shahrah-e-Faisal, Karachi

Technical Session 8 Water Resource Engineering **Time:** 11:15 to 1:15 pm **Venue: IEP Convention Centre** **Session Chairs** Engr. Farhat Adil, IEP Engr. Payam Siddiqi, IEP	**Technical Session 9** Structural Engineering Time; 11:15 to 1:15 pm **Engr. Asfia Aleem Hall** **Session Chairs** Engr. Dr. Masood Rafi, NEDUET Engr. Faiza Saeed, IEP	**Technical Session 10** Materials Engineering **Time:** 11:15 to 1:15 pm **Venue: Dr. S.A. Hassan Hall** **Session Chairs** Engr. Dr. Shuaib Ahmed, NEDUET Engr. Dr. Abdul Qudoos, BUE-Khuzdar	**Technical Session 11** Geotechnical Engineering **Time:**11:15 to 1:15 pm **Venue: Seminar Hall** **Session Chairs** Engr.Dr. Sadaf Qasim, NEDUET Engr. Dr. Anil Kumar, MUET
Water Saving Techniques in Civil Engineering Department of Mehran UET Jamshoro *Haresh Kumar, Ashfaque Ahmed Pathan, Abdul Waheed Kalhoro, Jaipal Rathore and Kelash Kumar*	**SFRC specimens under increasing compressive loading rates** *Laiba Ayub, Minza Mumtaz and Shamsoon Fareed*	**Analysis of Adhesion and Moisture Susceptibility of Different Modified Bitumen Using Bitumen Bond Strength and Rolling Bottle Testing Techniques** *Muhammad Sohail Jameel, Naveed Ahmad, Syed Bilal Ahmed Zaidi, Hafiz Ammar Zahid, Sohail Iqbal and Muhammad Tausif*	**Soil reinforcement by using geotextile** *Asif Nazir, Usama Khan, Rana Muhammad Umar Farooq and Muhammad Abrar*
Integration of Low Head Turbine with Wastewater for Power Generation: A Case Study *Musab Waqar, Hammad Khalid and Ibtihaj Ahmed*	**Influence of Asymmetry on Local Damage Response of Plan-Asymmetric Reinforced Concrete Structure** *Zeshan Alam*	**Use of admixtures in improving bonded asphalt overlay on concrete pavements-A critical review** *Minhas Shah, Muhammad Usman Farooqi and Majid Ali*	**Treatment of expansive soil by using silica fume** *Asif Nazir, Usama Khan and Rana Muhammad Umar Farooq*
Study of the flow characteristics around permeable and impermeable spur dike in an open channel *Sohail Iqbal, Ghufran Ahmed Pasha and Usman Ghani*	**Experimental Research on Post-Fire Repair of RC T-Beams Using Method of Curing** *Salman Khan, Muhammad Yaqub, Muhammad Noman and Muhammad Usman Rashid*	**Assessment of asphalt overlay performance over concrete bridge deck** *Syed Zulnorain Ali, Majid Ali and Usman Farooqi*	**Foundation on Swelling Soils** *Farooq Fazal*
Effect of submerged vegetation with increasing patch density on the flow structure *M. Hasnain Tariq*	**Behavior of interlocking plastic-block wall with opening under harmonic loading using locally developed shake table** *Mehran Sudheer and Majid Ali*	**Comparative study of concrete properties using different coarse and fine aggregate abundantly available in Pakistan** *Asif Nazir, Usama Khan and Rana Muhammad Umar Farooq*	**Evaluation of Fatigue Characteristics of Reclaimed Asphalt Pavement (RAP) Binder** *Muhammad Hammad Hassan*
Invited Talk by **Engr. Noorul Ayen Hasan**	**Invited Talk by** **Engr. Muhammad Monis Suri**	**Invited Talk by** **Engr. Hasan Akhter**	**Invited Talk by** **Engr. Saif Ahmed Saeed**

ICEC 2020 Parallel Technical Sessions
Day 2- Saturday, March 14, 2020
PARALLEL TECHNICAL SESSIONS
Time 2:00 - 4:00 pm
Venue: IEP Convention Center, 5th Floor, IEP Building, Shahrah-e-Faisal, Karachi

Technical Session 12 Structural Engineering **Time**: 2:00-4:00 pm **Venue: Engr. Asfia Aleem Hall** **Session Chairs** Engr.Prof. Dr. Rashid A. Khan, NEDUET Engr.Asfia Aleem, IEP	**Technical Session 13** Materials Engineering **Time**: 2:00-4:00 pm **Venue: Dr. S.A. Hassan Hall** **Session Chairs** Engr.Dr.Amanullah Marri, NEDUET Engr. Manzoor Nadeem, IEP	**Technical Session 14** Structural Engineering **Time**: 2:00-4:00 pm **Venue: Seminar Hall** **Session Chairs** Engr. Prof.Dr. Asad ur Rehman Khan, NEDUET Engr.Sartaj Ahmad Khan, IEP	**Technical Session 15** Infra-structure Engineering **Time**: 2:00-4:00 pm **Venue: IEP Convention Centre** **Session Chairs** Engr.Dr. Adnan Qadir, NEDUET Engr. Nooruddin Ahmed, IEP
A critical review on limitations of non-linear analysis of multi-storey structures *Asma Bashir, S ana Gul and Majid Ali*	**Effect of Rice Husk Ash on Properties of Self Compacting Concrete Containing Marble Powder** *Muhammad Hamza Irshad and Ayub Elahi*	**Energy Dissipation in 3D Structure With Multiple Uplifts at Column Bases** *Hassam Ud Din and Majid Ali*	**Analyzing the mobility of multi-lane highway using travel time as a performance measure** *Jamal Khan, Muhammad Khurshid, Arshad Hussain and Muhammad Iqbal*
Influence of Jute Fibre and GFRP Rebars in Compressive and Dynamic Behavior of Prototype Thin Shear Concrete Walls *Abaid Ur Rehman and Majid Ali*	**Effect of Heavy Weight Magnetite Aggregate on Mechanical and Radiation Shielding Properties of Concrete** *Muhammad Umair Anwar and Ayub Elahi*	**Performance Comparison of Circular and Rectangular Cross-Sectioned FRP Stirrups** *Muhammad Tahir and Zhenyu Wang*	**PERFORMANCE EVALUATION OF MRF UNDER REAL AND ARTIFICIAL GROUND MOTIONS: A Study of Quetta Region Baluchistan** *Naik Muhammad, Farhad Khan, Muhammad Umar, Abdul Samad, Muhammad Ibrahim, Najeebullah*
Identification of cracks in concrete by visual inspection and recommendation for their repair - case studies *Kapil Dev*	**Effect of Waste Glass Powder on the Fresh and Hardened Properties of Concrete** *Naraindas Bheel, Santosh Kumar Meghwar, Zubair Hussain Shaikh, Rameez Ali Abbasi, Irfan Ali Shar, Ali Aizaz Dayo*	**Out-of-plane behavior of mortar-free interlocking solid-walls under seismic loading – A review** *Sohail Afzal and Majid Ali*	**Seismic Analysis and Design of Elevated Water Tower – A Review** *Muhammad Bahram Tufail Kiani and Majid Ali*
Finite element modelling of uplift in structures - a review *Irfan Waris and Majid Ali*			
Invited Talk by **Engr.Ahsan Nadeem**	**Invited Talk by** **Engr. Al-Kazim Mansoor**	**Invited Talk by** **Engr. Sadaf Fatima**	**Invited Talk by** **Engr.Dr. Ashar Lodi**

Full Length Papers

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Seismic Performance of Moment Resisting Reinforced Concrete Frames under Code Compatible Ground Motions

Naik Muhammad, Abdullah, Faiza Nadeem, Mahnoor Mirwani, Sahibzada Shah Muttal, Abddul Munaim, Behroz Khan, Saeed Ullah Jan Mandokhail
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Abstract
Most areas of the country, Pakistan, lie in Zones 2B, 3 and 4 (as per UBC'97) making them prone to seismic events of high intensity. Even if only the last 100 years are considered the damage caused by such hazards can be highlighted in red. The increase in population of the country further increases the risks of damage caused by any such future event. Therefore, it is highly significant to understand the seismic vulnerability of the country. As the structural stability mostly depends on building frames hence it is necessary to consider their seismic performance while designing. Even though the frames are designed according to the building codes of the region, however, they are not followed 100 % which necessitates the investigation of the seismic performance of such structures. This paper presents dynamic analysis of a code compliant reinforced concrete moment resisting frame in accordance to BCP-2007, under real ground motions; which has already been analyzed experimentally using shake table tests. The purpose is to carry out a comparative study between experimental and theoretical dynamic analysis.

Keywords
Seismic Analysis, Moment Resisting Frames, Code Compatibility, UBC-97, BCP-2007

1. Introduction

The vulnerable seismicity around the globe has become an important factor to be taken under consideration when designing structures especially where ground motions are frequent. One of the major causes of massive causalities and economic loss during a seismic event is the failure of structural assemblies, therefore various studies have been carried out to design and build structures that can resist such lateral loads. The extent of protection provided by these structures depends upon seismic intensity and economic limitations. Practically, limit states are considered for the providence of seismic protection to the structures, which are related to preservation of functionality, minimizing the damage to the building and avoidance of loss of life. Seismic performance of a structure is based on the concept of acceptable levels of damage under one or more than one events of specified intensity (Kashyap, 2009).
Structures can be built that can withstand extreme earthquakes but they may end up being uneconomical. In order to achieve the economical construction of earthquake resistant reinforced structures, relevant

computational and experimental studies have been carried out that have led to the development of seismic design codes for multi-story reinforced concrete structures (Seo, Hu, & Davaajamts, 2015).

Seismic Provisions, initially based upon wind load and static force concept, became a part of Uniform Building Code in 1927. The equivalent static force procedure remained the standard requirement by 1940s (Housner, 1984). Earthquakes, however are dynamic, inelastic and random in nature rather than static. Various studies that were conducted, have shown that static analysis may be sufficient for low to medium rise structures lying in Zone 1, 2A and 2B while dynamic analysis is required for seismic safety of high-rise structures (Bagheri, Firoozabad, & Yahyaei, 2012) (Gottala, Kishore, & Yajdhani, 2015) (Kakpure & Mundhada, 2016).

In 1943, structural dynamics were considered and the seismic co-efficients were indirectly related with number of stories and then directly in 1956. The modal response spectrum was introduced in 1957. The effect of inelastic energy dissipation was first considered in 1959 but it still not fully developed up until 1970s (Fajfar, 2018). Time History Analysis is the advanced method of analyzing seismic response, and provides a higher safety factor as compared to linear analysis (Patil & Kumbhar, 2013).

Currently, different analysis procedures are available having different complexity levels to evaluate seismic responses of structures. If applied with appropriate design, dimensioning and detailing, they would effectively increase the structures' stability during a seismic event. The Analysis Based Design methods have enabled the design of structures with increased life safety but in most cases the structures still fail to keep their functionality. The resulting refurbishing costs are economically straining. For this purpose, Performance Based Design Methods are taken into account-in order to increase the seismic performance of new structures as well as already built structures.

In order to test and improve the effectiveness of seismic provisions, many engineers and scientists have worked-designing, analyzing and assessing structures that were expected to be under seismic risk. Joseph M. Bracci et al. studied the behavior for 3-storey 1:3 gravity load designed model frame. They resisted minor earthquakes without major damage but suffered extensive sideway deformation under severe earthquakes (Bracci, Reinhorn, & Mander, 1995).

Lee et al. analyzed a 1:5 scale, 2-bay 3-storey RC frame model designed in accordance to the Korean practice of non-seismic designing under various earthquake magnitudes. The structure not only resisted design level earthquakes but also higher seismic intensities (Lee & Woo, 2002). M. Rizwan et al. experimentally studied the seismic behavior of five 1:3 scale 2-storey representative models by performing shake table tests. Rizwan's paper particularly highlighted that the decrease in structures' code compatibility lessens the R factor which causes the drift to increase. He also concluded that ACI 318 requirement for exterior column depth of 15 times diameter of longitudinal bars is inadequate to avoid joint panel damage particularly for non-compliant frames, under design level earthquakes (Ahmad et al., 2019).

Pakistan is a region with high seismic risk as it lies on active faults having high seismicity. Since 1904, hundreds of earthquake shocks of magnitude 4 and above have been instrumentally recorded. Amongst these, the most damaging ones are Kachhi (1909), Sharigh-Mach (1931), Quetta (1935), Makran (1945), Duki (1966), and Pattan earthquakes (Kazmi, 2006). The October 2005 Kashmir earthquake in particular, with a magnitude of 7.6 shook about 450,000 buildings resulting in as much as 72763 life causalities. Seismic Provisions were introduced in Pakistan Building Code in 2007 based on UBC-97. However, it still carries a lot of flaws (Rossetto & Peiris, 2009).

Quetta, with a population of about 1.001 million (2017), is one of the most seismically active zones of the country, and has suffered several intense earthquakes throughout its history. The 1935 earthquake, recorded 7.7 lasted for three minutes destroyed almost the entire assemblies resulting in over 35000 fatalities (Erduran, Magsi, Gill, & Lindholm, 2015). Numerous other human-felt earthquakes have been

recorded. Despite being prone to high seismic risks, the structural designs do not comply with the seismic code requirements of the area. This severely increases the possibility of mass casualty and economic loss during a seismic event. Consequently, it is highly important to design code compatible seismically resistant structures. In addition to that, studies need to be conducted to evaluate the seismic response of structures compatible to seismic provisions of UBC'97 under real ground motions so that the existing flaws in BCP'07 can be rectified to minimize future losses.
The objective of this study is to evaluate the behavior of reinforced concrete moment resisting frame when subjected to real ground motions. For this purpose, reinforced 3D frame compatible with UBC'97 is analyzed under gravity loads and real ground motions. The resulting roof drift is then compared with the previously available experimental results of shake table tests.

1.1. Methodology

In this study, a frame compatible to UBC'97 is selected and analyzed for dynamic responses using Finite Element Analysis under real ground motions. The responses are then compared with the already available experimental results. The purpose is to conduct a comparative study between the results of experimental analysis and finite element analysis. The model selected is a 2-bay by 1-bay and 2-story structure originally tested by Rizwan et al using shake table tests (Ahmad et al., 2019). The input parameters are shown in Table.1, the 3D physical shape, plan, front and side elevations are shown in Fig. 1, 2 and 3.

Table 1- Input Parameters of 3D Frame

Frame	**Members dimensions (in)**	fc'	fy	**Dead Load (lb/ft²)**	**Live Load (lb/ft²)**
Two bays x one bay center to center distance of 18 feet and story height of 12 feet	Beam 12 x 18	3000 psi (21 MPa)	60 ksi (414 MPa)	40 (1st floor)	60 (1st floor)
	Column 12 x 12			60 (Roof)	40 (Roof)

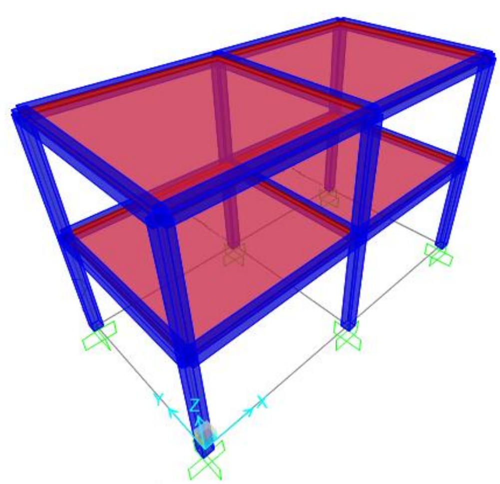

Figure 1- 3D Frame

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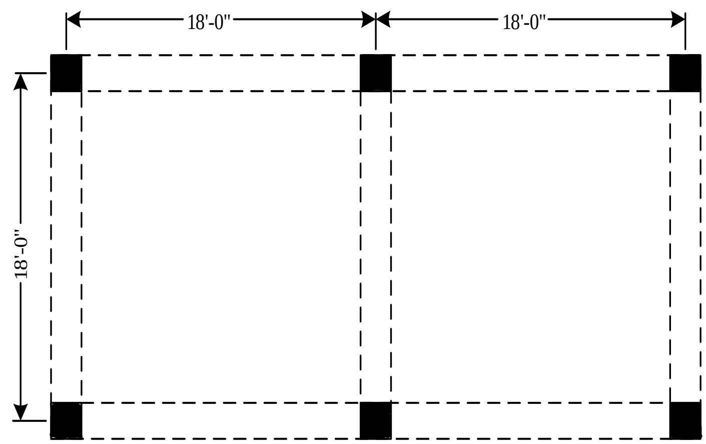

Figure 2- Plan of 3D Frame

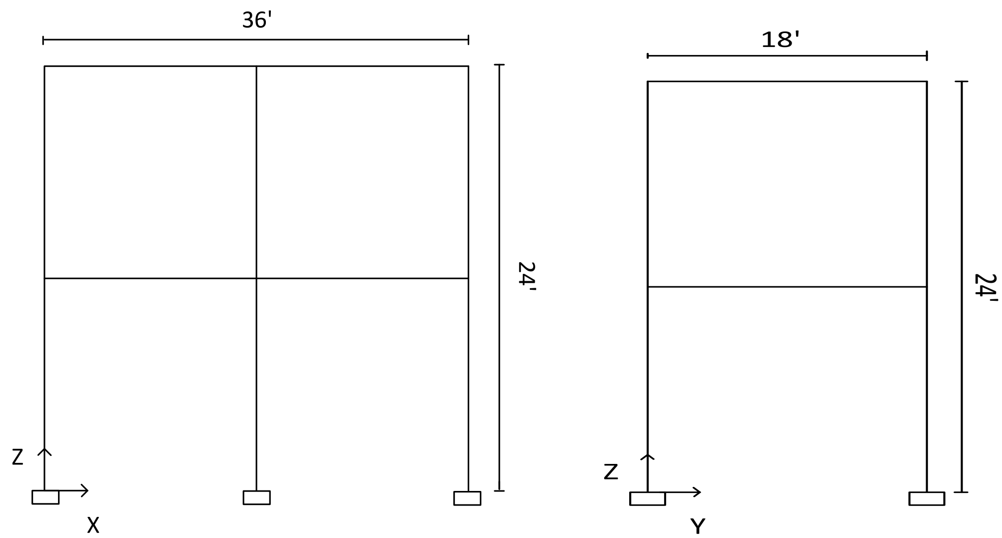

Figure 3- Front and Side Elevation of 3D Frame

The foundation of the frame has fixed supports with restrained moments and forces in all directions. The internal forces and displacements are then found by subjecting it to real ground motion.

1.2. Ground Motion

For dynamic analysis the real ground motion of 1994 Northridge earthquake is selected, as the Shake Table Tests were conducted using the same data. Hence, for comparison of results, this data is most suited. Fig 4 shows the accelerogram for 1994 Northridge Earthquake.

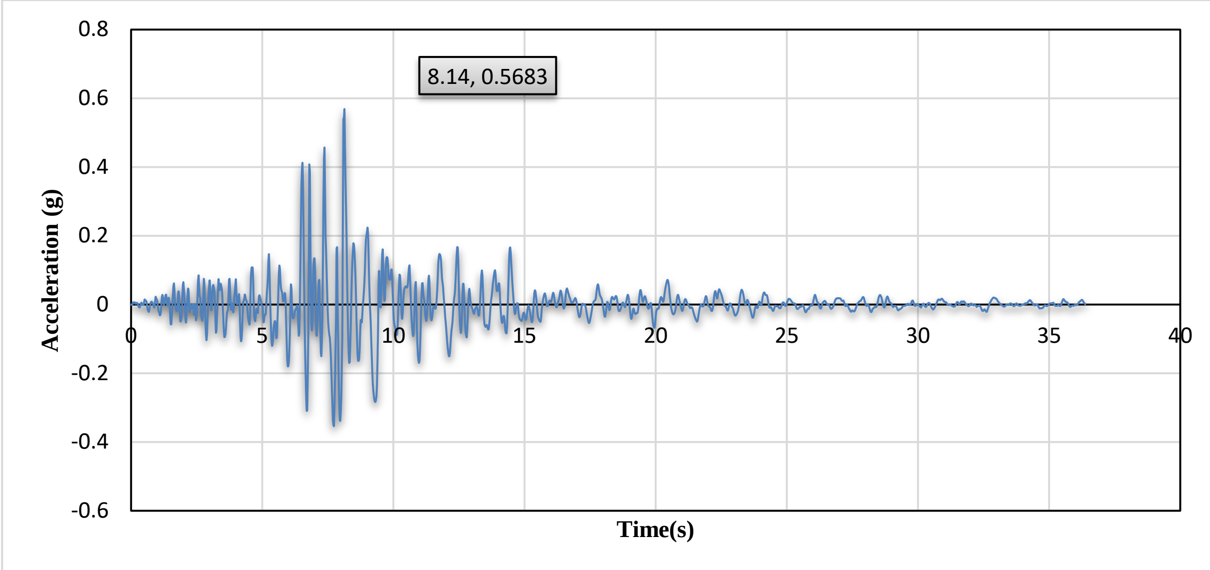

Figure 4- 1994 Northridge Ground Motion

2. Results

Joints and members output based on dynamic analysis:

The resulting reactions, displacements and internal forces of the specified frame, when subjected to the real ground motion, are illustrated in the following figures. Table 2 shows the joint reactions, while fig. 5, 6, 7 and 8 show the displacement of joint 12, displacement of joint 11, shear force of frame member 20 and bending moment of frame member 20 respectively.

Table 2- Resulting Joint Reactions

Joint	Output Case	X or F1 (lb)	Y or F2 (lb)	Z or F3 (lb)
1	Linear static	187.24	-175.08	39849.68
4	Linear static	-3.23E-14	-161.54	72850.64
7	Linear static	-187.24	-175.08	39849.68
10	Linear static	-187.24	175.08	39849.68
13	Linear static	1.312E-14	161.54	72850.64
16	Linear static	187.24	175.08	39849.68

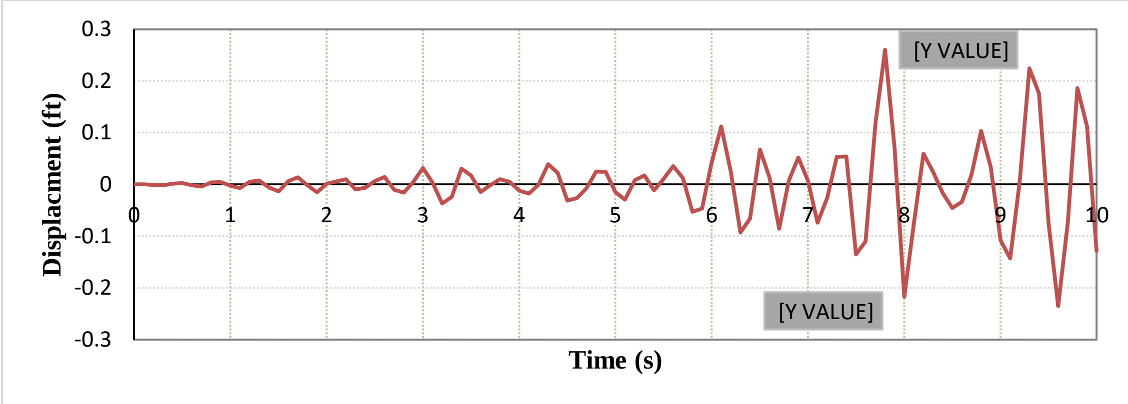

Figure 5- Joint 12 Displacement in X-direction

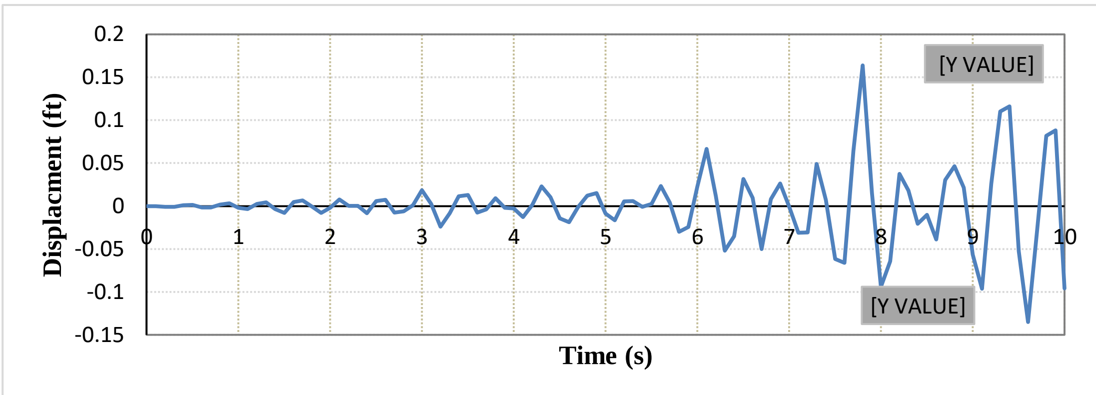

Figure 6- Joint 11 Displacement in X-direction

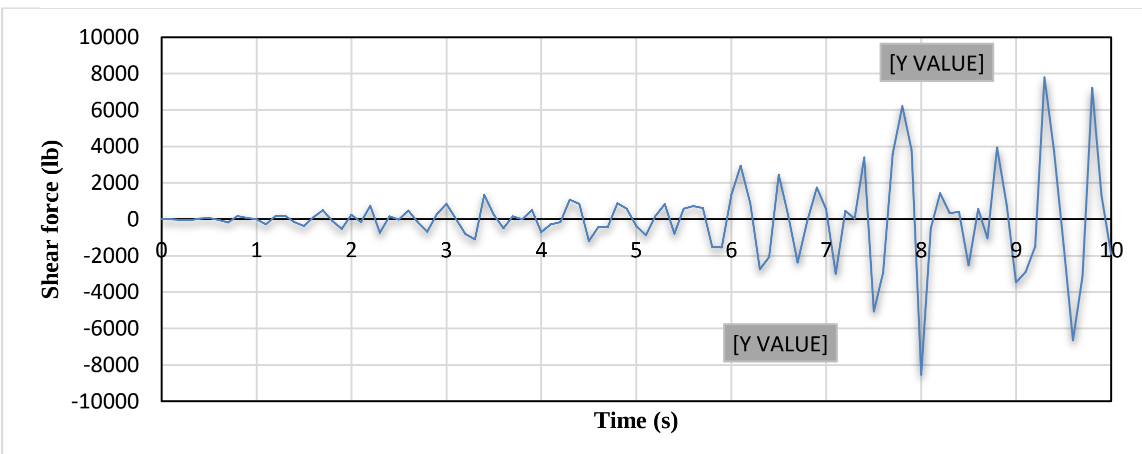

Figure 7- Shear Force on Frame Member 20

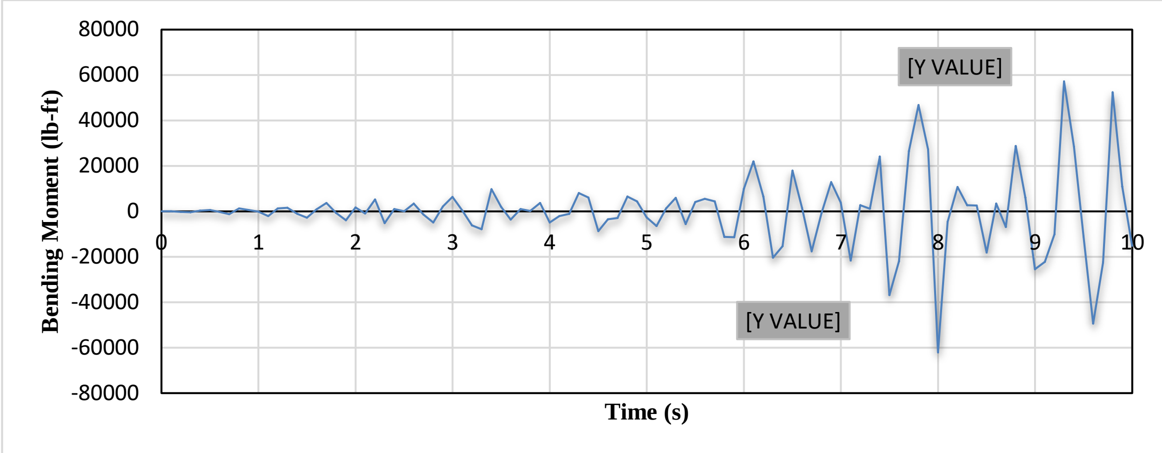

Figure 8- Bending Moment of Frame Member 20

Roof displacement of experimental analysis:
The results of shake table tests show a roof drift of 2.42mm and 0.87%.

Comparative Analysis:
The comparison between resulting roof displacement and drift at joint 12 in x-direction found by experimental analysis and finite element analysis is illustrated by Fig 9 as follows:

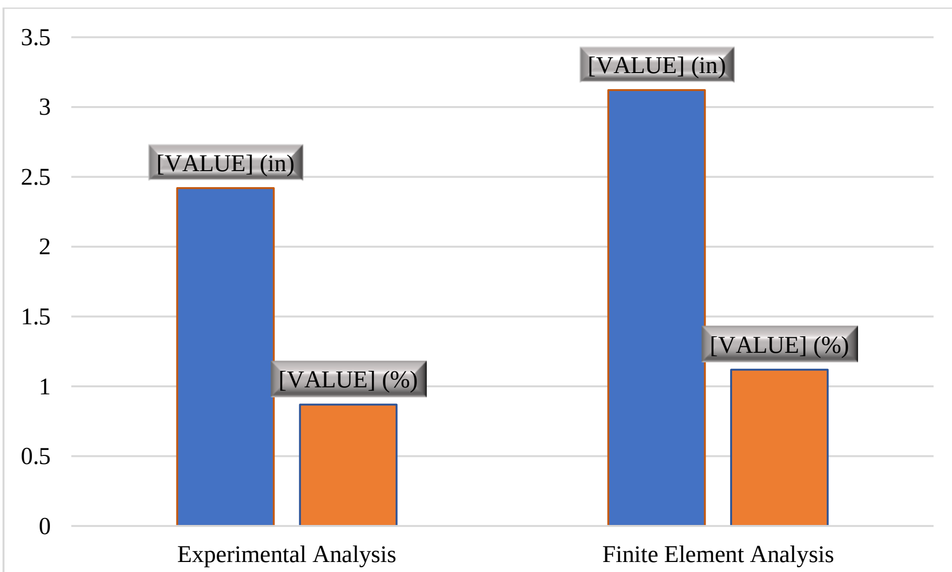

Figure 9- Comparison of Roof Displacement and Roof Drift (%)

The resulting roof displacement from experimental analysis is 2.42in while the resulting roof displacement obtained when the frame is subjected to finite element analysis is 3.12in. From the resulting roof displacement, the roof drift of experimental analysis and finite element analysis is 0.87% and 1.12% respectively.

3. Conclusion

The results obtained when the specified frame is subjected to dynamic analysis through shake table tests and finite element analysis show a difference of 0.7in in roof displacement while 0.25% in roof drift, which shows that the results of theoretical dynamic analysis deviate about 29% from the results of experimental dynamic analysis. However, as finite element analysis provides a higher value of roof displacement and roof drift, hence it can be used for a more conservative structural design.

References

Ahmad, N., Shahzad, A., Rizwan, M., Khan, A. N., Ali, S. M., Ashraf, M., . . . Alam, B. (2019). Seismic Performance Assessment of Non-Compliant SMRF-Reinforced Concrete Frame: Shake-Table Test Study. *Journal of Earthquake Engineering, 23*(3), 444-462.

Bagheri, B., Firoozabad, E. S., & Yahyaei, M. (2012). Comparative study of the static and dynamic analysis of multi-storey irregular building. *World Academy of Science, Engineering and Technology, 6*(11), 1847-1851.

Bracci, J. M., Reinhorn, A. M., & Mander, J. B. (1995). Seismic resistance of reinforced concrete frame structures designed for gravity loads: performance of structural system. *Structural Journal, 92*(5), 597-609.

Erduran, E., Magsi, A., Gill, A., & Lindholm, C. (2015). Earthquake risk assessment of Quetta. *Kjeller, Norway, Available online: http://www. pmd. gov. pk/seismic/3. pdf, Accessed, 26*.

Fajfar, P. (2018). *Analysis in seismic provisions for buildings: past, present and future.* Paper presented at the European Conference on Earthquake Engineering Thessaloniki, Greece.

Gottala, A., Kishore, K. S. N., & Yajdhani, S. (2015). Comparative study of static and dynamic seismic analysis of a multistoried building. *IJSTE-International Journal of Science Technology & Engineering, 2*(01).

Housner, G. (1984). An historical view of earthquake engineering. *Revista de Ingeniería Sísmica*(31), 1-16.

Kakpure, G. G., & Mundhada, A. R. (2016). Comparative Study of Static and Dynamic Seismic Analysis of Multistoried RCC Building by ETAB: A Review. *International Journal of Emerging Research in Management &Technology, 5*, 17-20.

Kashyap, J. (2009). *Seismic performance of reinforced concrete frames.*

Lee, H.-S., & Woo, S.-W. (2002). Seismic performance of a 3-story RC frame in a low-seismicity region. *Engineering Structures, 24*(6), 719-734.

Patil, A., & Kumbhar, P. (2013). Time history analysis of multistoried RCC buildings for different seismic intensities. *International Journal of Structural and Civil Engineering Research, 2*(3), 194-201.

Rossetto, T., & Peiris, N. (2009). Observations of damage due to the Kashmir earthquake of October 8, 2005 and study of current seismic provisions for buildings in Pakistan. *Bulletin of Earthquake Engineering, 7*(3), 681-699.

Seo, J., Hu, J., & Davaajamts, B. (2015). Seismic performance evaluation of multistory reinforced concrete moment resisting frame structure with shear walls. *Sustainability, 7*(10), 14287-14308.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Fire Protection and Safe Evacuation Issues for Congested Areas in Pakistan

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Abstract

The restoration of fire-damaged structures is a subject in which most of the architects and structural design engineers have very little or no experience. Moreover, fire accidents are common in Pakistan and frequently observed in residential and commercial buildings. Nearly 720-fire incidents reported in the first three months of the last year. The increasing number of fire accidents in the metropolitan city is an alarming sign. During construction, few or no safety provisions are being considered against fire. No evacuation plan, smoke detectors, fire extinguishers are being planned or installed in residential buildings. Due to developing nation, Pakistan must take an alarming view of this hazard, which is almost neglected, resulting into massive property and life losses in individual accident. In this paper, some major issues are being highlighted together with recommendations which can be easily implemented to ensure the safety of the inhabitants within the budget. The recommendations also can be used to upgrade laws of building control authorities and to take corrective action for changing the system, to prevent life losses and property damages.

Keywords
Accident, Building, Evacuation, Fire, Safety, Structure

1. Introduction

The restoration of fire-damaged structures is a subject in which most of the architects and structural design engineers have very little or no experience. On the other hand, increasing number of fire accidents have shown major knowledge and mitigation barriers to solve common industrial and domestic fire problem. Now a day the industry and internet revolution is based on Cyber-physical system (CPS) to deal with any emergency alerts. This will support the concept of a smart city and smart firefighting initiative in the developed countries. However, developing countries are still using the conventional technique to deal with fire accidents. According to WHO approximately 180,000 deaths are caused by fire burns every year in low-middle income countries (WHO, 2018). In Pakistan, houses and apartments are expensive and the average person can spend their entire life saving to construct or buy a single house. House related fire accidents are some of the most devastating accidents occuring in a peoples lives. The effects can be tragic, in terms of massive losses of lives and finances. Fire accidents cause financial instabilities when entire life assets burn in a fraction of time. In such stressful conditions, high priority should be given in the country to minimize number of accidents. Pakistan is a major importer of Fire & Safety Products, crossing the import by 1 Billion US Dollars and the demand is growing further. The Government along with private sectors has an aim to bring in the latest technologies in Pakistan to ensure the safety of the Country (Fire and Safety Asia 2019). Despite product manufacturing, only in Karachi, roughly 720 fire incident has been reported in the first three months of 2019 (Tribune, 2019). The lack of awareness of safety measures leads to a big disaster such as no provision for fire exit, poor electric wiring, and even expired fire extinguisher are common causes of large fire accidents. For instance, expired fire extinguishers were found during a fire accident at Awami Markaz, which was the main hindrance in surpassing fire at first place as shown in Fig 1.

Figure 1: Fire Extinguisher discovered during Sep 2017 Fire Accident at Awami Markarkaz Islamabad (Tribune, 2017)

The electric circuit is another big reason behind most of the fire in offices, home, factories. Poor electric load distribution, improper inspection and maintenance of electric appliance on unattended cause sudden fire outbreak. Furthermore, a small project of 1000 - 400 sq. yards area is usually built with no provision for emergency exit. Most of the old office buildings are sans emergency exit. An example is Noor Trade Centre fire accident in March 2019. In new mega projects more attention is given towards safety provisions than in small projects. Now the question is what should be minimum safety standards and precautions that could potentially avoid fire accidents when we have congested city environment?

At the first stage, academic and industry should work on awareness campaign. After fire awareness and problem identification, the next stage would be introducing product testing, fire safety equipment in buildings. Accident investigation will be promoted and encourage researcher and engineers to work on Fire modelling for a real accident as a lesson learnt. Construction policy, architectural planning, town planning and building code will further develop for emergency conditions. Material research under fire such as paint, reusable construction material, household item will be tested and approval before using in building to avoid toxicity during fire accidents. Currently in Pakistan and most of the developing countries, do not have provision for fire safety and lack of awareness leads to catastrophic disaster and life losses. Particularly, large spaces like auditoriums, cinema halls, office building and universities which used heavy equipment and furniture without considering worse effects of these materials as fire fuel and toxicity.

The objective of this paper is to overview the safety condition in Pakistan particularly in Karachi. This paper highlights topics such as human behaviour, current building conditions, structural / material response, recommended NDT tests for concrete and steel after the fire exposure together with possible solutions.

2. Human Behaviour

In case of fire as heat flux and smoke affect the human body during panic and suffocation sometimes people jumped from high-rise buildings when unable to hold on or bear the heat as happened in past (Shakeel, 2017). There is no proper sign for people entering any building particularly most of the office buildings. New, therefore, it is a challenge to evacuate building efficiently in case of any emergency. The similar, case was observed when an intern student from Iqra University trapped in Noor Trade (14-storey building) in Gulshan-e-Iqbal, Karachi. During the fire accident, the student was on the seventh floor. The student was somehow managed to come alive but due to inhaling toxic gases with 37% body burn she succumbed her wounds and passed away the same night. During the fire accident, several people were injuries due to panic and stampede. During the same incident, two men jumped from high-rise building due to panic and lost hope of the emergency response team. Years after a similar fire accident in Saddar area happened when a young man lost his life after trying to escape from 10th storey building (State Life Building) (Khan, 2019).

Figure 2: Noor Trade Centre Karachi Fire Accident (Khan, 2019)

Despite emotional individual response another aspect is cultural differences and taking no concern to safety. To observe the human response in case of an emergency. A short experimental fire drill was organized to observed human response in an academic institute while it was fully occupied. Total 1078 students and staff were present at the time of fire alarm. Mostly student and staff used the main staircase to exit building only 6 to 10 students used the second emergency exit. This is because it is human phycology to choose a route, which is most similar to them in case of emergency when emergency drills are not practised. People did not like to stop what they were doing and neglect alarm by considering fault, as they are not trained to react immediately.

Even after 10 minutes of fire alarm and sending security guards to inform about the fire alarm, only 20 % responded and evacuated the building while 80% students and staff did not respond to fire alarm and followed gauds instructions. Only 20-30% of staff fully participated in a fire drill while others consider it time-wasting activity.

Currently, household materials are being used without testing for their potential hazard when exposed to fire. People buy house hold items with out awareness or concern about the potential fire hazard. Majority of deaths during fire accidents are being reported due to inhaling toxic gases, which are generated from paints, plastic items, furniture, home decoration items etc. Therefore, there is a need to improve and enhance safety awareness and testing commonly used household items before buying for the home. Secondly, it should be compulsory for any commercial and residential building to clearly show the fire exit route plan at each floor.

3 Urban Planning and Building Conditions

Most of the area is extremely congested in Karachi with less or no provision for emergency response. Areas such as Gulshan, Gulistan-e-Johar, Saddar, Nazimabad, Lalukehat, FB Area, Malir etc. are some of the densely populated and congested areas. In short, Karachi infrastructure is facing the most difficult situation, as there is insufficient firefighting capacity, lack of infrastructure and inadequate firefighting stations. Current firefighting capacity cannot cover floors more than 20 storey building. Water shortage is also a challenge together with narrow streets and poor master plan. Below are the typical cases and situation of metropolitan city Fig 3 to Fig 6.

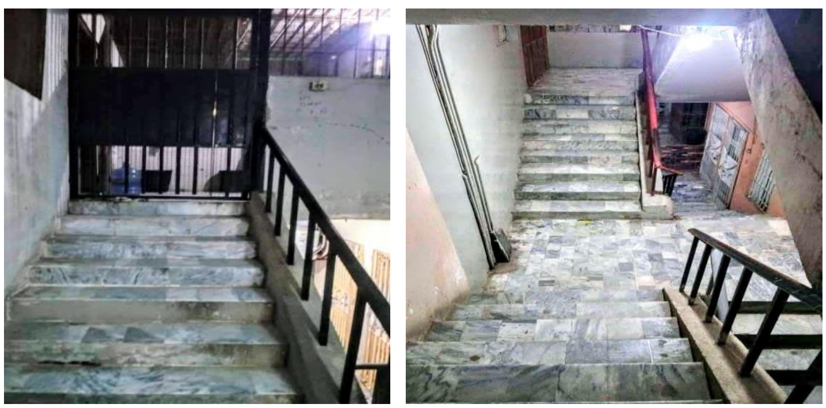

Figure 3: Narrow Staircase

Figure 4: Roof Locked with Single Congested exits

Figure 5: Congested Areas with Tangled Electric, Cable and Telephone Wirings

Figure 6: Joint Commercial and Residential building with Single Access

4. Material Response

After the fire accident, to assess the structure the following are the procedure as shown in g Fig 7. Heat changes the physical and chemical properties of any construction material concrete. During fire accident despite the high fire, resistance concrete undergoes various phase change depending upon the type and duration of fire exposure. The mechanical and physical properties at elevated temperature affect the compressive and tensile strength of concrete. Concrete when exposed to fire loss its moisture when exposed to fire. The intact moisture dehydrates and forms micro-crack by developing pore pressure during evaporation. The concert spalling may also be observed when temperature rise is sudden spalling is caused by high pore pressure. Embedded steel on the other hand loses its yield strength rapidly at 400-550 °C (M Imran et al., 2018; Muhammad Imran et al., 2018). Between 600 to 900 °C, the lime-stone undergo decarbonation. Concrete cubes heated up more than 1000 °C disintegrate and spalling occurrs from corners. After continuous exposure of 60 minutes, the strength of concrete is reduced by 35% - 55% of the normal strength of the concrete without fire exposure (Abdulkadir, Karim, & Abdullah, 2017).

Steel, on the other hand, has no strength reduction up to 400 °C however after 400°C in most of the cases steel contribute nothing (M Imran et al., 2018; Muhammad Imran et al., 2018). At 650

°C only 20% strength is left. Hot-rolled bar loss it elastic modules above 100 °C and yield plateau will disappear above 200 °C. However, cold drawn bars are even more sensitive than a hot rolled bar. At 400 °C 50% strength reduced and at 650 °C only 10% strength remains (Albrektsson, Flansbjer, Lindqvist, & Jansson, 2011).

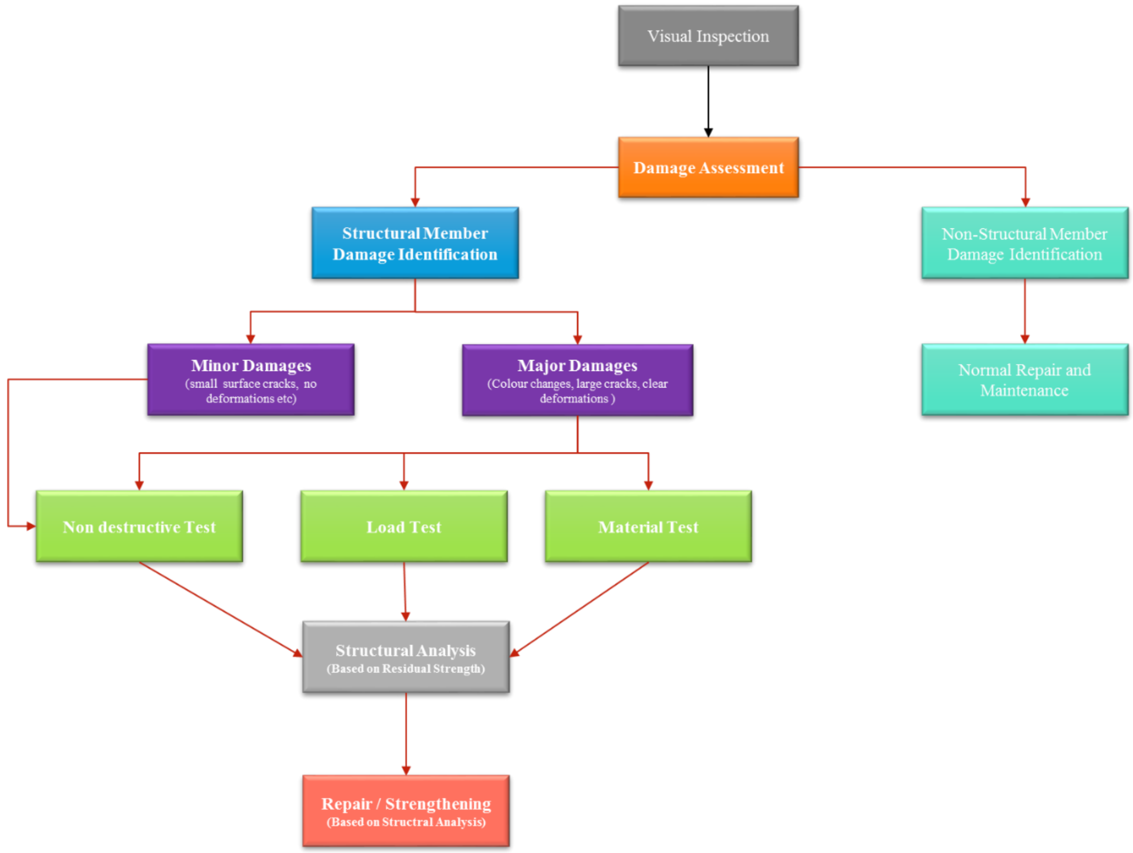

Figure 7: Proposed Flow Chart for Damage Assessment

5. Non Destructive and Destructive Tests after Fire Accidents

After fire accident or high heat flux exposure, it is important to assess the residual strength of structural the exposed structural member and compared with strength (Abdulkadir et al., 2017). Non-destructive tests are commonly used to assess the quality and integrity of a structural member after long service life or due to sudden damage without affecting its function as a structural member (Albrektsson et al., 2011) (Shah & Kaur, 2019; Zawawi et al., 2019). Particularly after fire accidents, followings are the recommended NDTs:

5.1 Visual Inspection: it is by far the most common method used to check preliminary inspection before deciding to select any other method. In the technique, the condition of the affected area is assessed with severe, moderate or no damage identification (Zawawi et al., 2019). Any sort of cracks, colour changes, spalling, horizontal or vertical crack are being identified. In the case of steel structure, only steel deformation can be assessed however difficult to identify material property degradation.

5.2 Ultrasonic Pulse Velocity: This technique generates data by sending stress waves and reflecting longitudinal ultrasonic waves. The pulse velocity assesses the quality of the sample by recording waves arrival time at the receiving surface (Colombo & Felicetti, 2007). Ultrasonic Pulse Velocity or UPV is the most trustworthy method to estimate static modulus of elasticity of fire-damaged concrete (Abdulkadir et al., 2017).

5.3 Rebound Hammer: Rebound Hammer (RH) test is conducted to assess the relationship between surface hardness and compressive strength of the concrete reproduced in the form of rebound number. The rebound values relate mainly to concrete condition at the near-surface layer, approximately to the depth of 3 cm (Awoyera, Akinwumi, Ede, & Olofinnade, 2014; Panedpojaman & Tonnayopas, 2018).

5.4 Digital Camera Colorimetry: The concrete colour changes when exposed to fire, however, the strength and colours changes depending on the type of aggregate and presence of more siliceous aggregate and less igneous and calcareous aggregate. Concrete turns normal to red or pink at 300 °C to 600 °C, whitish-grey 600°C to 900°C and buff 900°C to 1000°C (Awoyera et al., 2014). In this technique, digital images of thousands of pixel are compared by analysis of core extracted from an affected member.

5.4 Hardness Test: It is the most common test for steel when exposed to fire. This test is used to estimate the tensile strength of the steel member/bar and residual strength of the steel. There is a hardness test which can be tested either manually or automatically by applying appropriate load/shear force and measure the average diameter of the indentation (the Brinell Hardness Number).

5.5 Metallographic Examination: During the fire, steel chemical composition may be affected which cannot be determined without proper assessment of microstructural changes resulting from the fire. Details can be found in (Kirby, Lapwood, & Thomson, 1986). However, this test is expensive and required skilled personals.

6. Conclusion & Recommendation

There is a need to investigate the existing city condition and define the cities in different zones in order to identify the high-risk zone and improve accessibility for those areas

Common Observations

- Major issue or fire causes are unknown and no investigation has been performed to identify causes and learn lessons from past incidences.
- In most of the cases, a building fire extinguisher is expired and maintain not properly maintained on time.
- Emergency stairs cases are only in the paper during drawing approval and during construction, designated space removed.
- Due to congested and improper town planning, in case of any fire accident, it is difficult for the fire brigade to reach on time. Secondly, due to narrow lanes in most of the areas, there are open electric wiring, tangled telephone and cable lines make difficult to reach out on different stories of the building.
- Congested flats with no provision for safe exit or emergency evacuation route.
- Even big shopping centres do not have proper exit plans.
- Most of the balconies of the flats are closed permanently using grills or bricks.

- Congested shopping and residential areas such as Saddar bazar, Laoluketh, Mena Bazar, Empress Market etc always remained crowded without evacuation signs. Mostly shopping areas are build in the basement has an only single exit in case of emergency.

Possible Measures based on Observations (Recommendations)

When fire strikes, deadly smoke can fill in home or staircase within a minute. Following are the precaution during a fire for congested buildings/flats to safe life until fire brigade intervenes and surpassed fire.

- Always plan for two exits from the house/flat. If the first way out is blocked by thick smoke or fire, use second way out.
- In the case of Apartments/flats if the second way is a balcony and if the balcony is at height make sure a part of the grill must be open or can be open in case of emergency.
- Practice a fire drill at least twice a year. Try feeling your ways in the dark or with eyes closed. This practice with children and elderly people should be under supervision.
- Make sure all windows and door that lead outside can be easily open. No jam or furniture should block the emergency exist
- If the flat is in the congested area try to construct another stair linked to another building from the roof or have a separate stair from the roof to ground floor.
- If the second exit is blocked and first way out is filled with thick smoke than get low and go crawl quickly under the smoke to your nearest exit. Make sure close doors behind you to avoid fire spread rapidly.
- Connected roofs should be allowed in order to reduce building evacuation time from one property to another in case there is no emergency exist. In the absence of an emergency fire exit, the roof is the only emergency escape, which is in most of the cases found locked in most of the apartments.
- Never stay long for collecting precious items remember you can develop if you are alive.
- If your loved one trapped inside never, try to go back, stay outside call the fire brigade, and inform them possible location with few identical signs correctly.
- Draw a map of each level of your home showing all doors and windows discuss and plan evacuation with everyone who lives with you.
- In the case of old construction plan a new exit door and construct the low budget safe staircase.
- Sindh Building Control Authority (SBCA) should conduct a detail building survey to identify the potentially high-risk area for immediate action and make sure engineers, architect, contractors compliance SBCA rules and regulation.
- Emergency fire escape stair should be mandatory for all residential and commercial building and rules must be followed by KBCA, DHA etc.

7. Acknowledgement

I would like to acknowledge my final year students Mr. Salman Lateef, Mr. Sarfaraz Nawaz, Mr. Talha Habib and Mr. Zaid Habib for conducting survey for this ongoing project.

8. References

Abdulkadir, M. R., Karim, A. A., & Abdullah, A. H. (2017). Destructive and Non Destructive Strength Evaluation of Concrete Exposed to Fire.

Albrektsson, J., Flansbjer, M., Lindqvist, J.-E., & Jansson, R. (2011). Assessment of concrete structures after fire. In.

Awoyera, P., Akinwumi, I., Ede, A., & Olofinnade, O. (2014). Forensic investigation of fire-affected reinforced concrete buildings. *IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), 11*(4), 17-23.

Colombo, M., & Felicetti, R. (2007). New NDT techniques for the assessment of fire-damaged concrete structures. *Fire Safety Journal, 42*(6), 461-472. doi:https://doi.org/10.1016/j.firesaf.2006.09.002

Imran, M., Liew, M., Garcia, E., Nasif, M., Yassin, A., & Niazi, U. (2018). *Deformation and Heat Transfer on Three Sides Protected Beams under Fire Accident.* Paper presented at the IOP Conference Series: Earth and Environmental Science.

Imran, M., Liew, M. S., Nasif, M. S., Gracia, E. M., Danyaro, K. U., & Niazi, M. U. (2018). Thermal and Mechanical Response of Partially Protected Steel I-Beam under Fire. *MATEC Web Conf., 203*, 06009. Retrieved from https://doi.org/10.1051/matecconf/201820306009

Khan, F. (2019, 2019/03/22/). Two jump to death as 14-storey commercial centre catches fire. *The News International*. Retrieved from https://www.thenews.com.pk/print/446993-two-jump-to-death-as-14-storey-commercial-centre-catches-fire

Kirby, B., Lapwood, D., & Thomson, G. (1986). The reinstatement of fire damaged steel and iron framed structures. *British Steel Corporation, Swinden Laboratories, 1986*, 79.

Panedpojaman, P., & Tonnayopas, D. (2018). Rebound hammer test to estimate compressive strength of heat exposed concrete. *Construction and Building Materials, 172*, 387-395.

Shah, E. S. B., & Kaur, E. M. (2019). Strength Evaluation of Fire Affected Hospital Building by using Non Destructive Test.

Shakeel, Q. (2017, 2017/09/10/). Two killed as Awami Markaz building in Islamabad catches fire. *DAWN News*. Retrieved from https://www.dawn.com/news/1356819

Tribune. (2017, 2017/09/11/). Awami Markaz inferno: Question marks raised over rescue services | The Express Tribune. *Express Tribune*. Retrieved from https://tribune.com.pk/story/1502850/awami-markaz-inferno-question-marks-raised-rescue-services

Tribune. (2019, 2019/04/04/). Nearly 720 fire incidents reported in 2019 in Karachi | The Express Tribune. *Express Tribune*. Retrieved from https://tribune.com.pk/story/1943070/1-nearly-720-fire-incidents-reported-2019-karachi

WHO. (2018). *Burns*. Retrieved from https://www.who.int/news-room/fact-sheets/detail/burns

Zawawi, N. A. W. A., Liew, M., Alaloul, W. S., Shawn, L. E., Imran, M., & Toloue, I. (2019). *Non-Destructive Testing Techniques for Offshore Underwater Decommissioning Projects through Cutting Detection: A State of Review.* Paper presented at the SPE Symposium: Decommissioning and Abandonment.

11th International Civil Engineering Conference (ICEC-2020)
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March 13-14, 2020, Karachi, Pakistan.

Condition Assessment of an Aging Bridge: A Case Study

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Abstract
Structure condition assessments play a critical role in the effective management of bridge infrastructure. Bridge elements deteriorate over an extended period of time and the rate of deterioration is a function of various factors, that effect the life span of the bridge. Bridges are considered structurally deficient if significant load-carrying elements are in poor condition due to deterioration or damage. A bridge classified as structurally deficient is not necessarily unsafe, until a detailed condition assessment or a live load test is performed. It should also be noted that a number of factors make it difficult to fully and easily assess the current condition of the existing structural elements. These include both the limited documentation available and the presence of hard finishes in many areas.

A three tier approach, including seismic and structural evaluation, is performed to investigate the deficiencies and possible weakening of the bridge structure located in Karachi. Based on the three tier approach, the overall behavior of the bridge structure shows adequate strength to meet the WPHC and AASHTO 2006 code demands.

Keywords
Bridge condition assessment, Bridge inspection, Bridge health monitoring, Structural evaluation, Bridge management system

1. Introduction

With the increased demand placed on aging infrastructure, there is great interest in condition assessment of bridges. Bridges are capital-intensive assets which require systematic maintenance and preservation. Most of the roadway bridges in Karachi were built before the adoption of modern principles of sustainable planning and seismic design, and are approaching their design lifetime. The volume and loading of the heavy freight vehicles, which they are carrying, are considerably larger than anticipated at the time of their construction. In most cases, these bridges are structurally deficient and degraded due to the ageing effects and inadequately maintained. Reliable assessment of their safety to seismic and increased operational loads is therefore required before deciding on their optimal management.

Condition assessment essentially pertains to the check of compliance with Performance Goals and requires the definition and computation of Performance Indicators. The routine deterioration that bridges undergo causes a loss in the intended performance that, if undetected or unattended, can eventually lead to structural failure. The level of performance for any structure can be measured based on different investigative procedures such as visual observations, non-destructive testing techniques, structural capacity analysis, or a combination of these. If an early-stage deficiency is detected in the structure then a decision based on cost-effective rehabilitation can be made. Similarly, in absence of bridge maintenance or delay in repair may lead to heavy future costs or degraded assets.

2. Approach to Condition Assessment

To achieve the purpose of the Bridge Condition Assessment (BCA), the inspection must be as thorough as possible within engineering reason. It must also be documented in such a manner as to allow a proper scope of work to be determined and approved. During the preliminary screening phase, Tier-1, potential deficiencies of the structural elements and force resisting system are identified. Next NDE testing, Tier-2, is employed to quantify the information about the state of the material and its charcteristics, so that the information can be used with confidence in condition assessment and remaining life estimates of a facility. A BDMS tool is then applied to evaluate the current state of the bridge and the level of deterioation it has experienced. Finally, evaluation phase procedure referred to as Tier-3, analytical procedure, is then performed for further exploratory work and address the deficiencies found.

3. Condition Assessment Chanllenge

In Pakistan, only one bridge design code (WPHC 1967) has been published based on the US code of (AASHO 1961). The WPHC code has never been updated. It is evident from the seismic hazard map, **Figure 1** that a significant part of Pakistan falls within zones that have a moderate to high seismic risk, including large cities such as Karachi, Quetta, Gwadar, Peshawar, Abbottabad, Gujrat, and Islamabad. There are approximately 6,000 bridges on the national highways of Pakistan, 67% of which were constructed prior to the 1980s. (Ali *et al*., 2011) indicated that since the new PGA values in Building Code of Pakistan (BCP, 2007) are significantly higher than the recommended design values of 0.02 g–0.06 g found in the Code of Practice for Highway Bridges 1967, therefore, it is critical that the engineering community in Pakistan work to ascertain the safety and code compliance of bridges constructed before 2007, especially those constructed prior to 1986. This means that the bridges which are designed prior to 1979 would have either used 2%-6% of the weight as the lateral force value or would have adopted arbitrary PGA values.

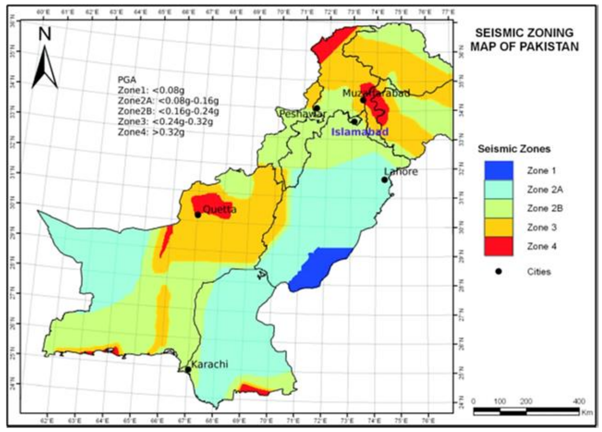

Figure 1: Seismic Hazard Map based on 475-year return period from the Building Code of Pakistan 2007. (BCP, 2007)

Furthermore, it is also important to note that in the older version of the AASHTO Standard of 1980s, (AASHTO 1980) simplistic elastic design procedures, i.e, the Equivalent Static Force Procedure and the Response Spectrum Method were allowed. However, the current AASHTO-LRFD (AASHTO 2006) method includes various analysis procedures ranging from elastic spectral analyses (Hwang et al. 2000), nonlinear static analyses (Basoz and Mander 1999, Shinozuka et al. 2000), and nonlinear time-history analyses (Shinozuka et al. 2000 and Choi et al. 2004).

4. Case Study Bridge

This bridge is designed by a UK based Consultant in 1964. The present day vehicular traffic mainly consists of heavy freight vehicles along with an assortment of smaller vehicles.

The total length of the structure is 2115’-2”, while the total length of the main bridge structure is 1145’-0’’ measured back-to-back of abutments with additional east and west ramp measuring 544’-5” & 425’-9” respectively, **Figure 2**. The bridge consists of multiple continuous spans (3-span continuous). Furthermore, the main span of the bridge is 465’-5” long, with an overall carriageway width of 54’-0”, including 6’-0” wide footpath on both sides. The deck-slab is 8” thick with a 1.5” thick overlay. Eight (8) pre-stressed beams spaced at 6’-9”, support the deck, **Figure 2**. Furthermore, the carriageway width near the abutment and at ramps is reduced to 46’-0” supported on seven (7) girders spaced at 6‘-9”, **Figure 2**.

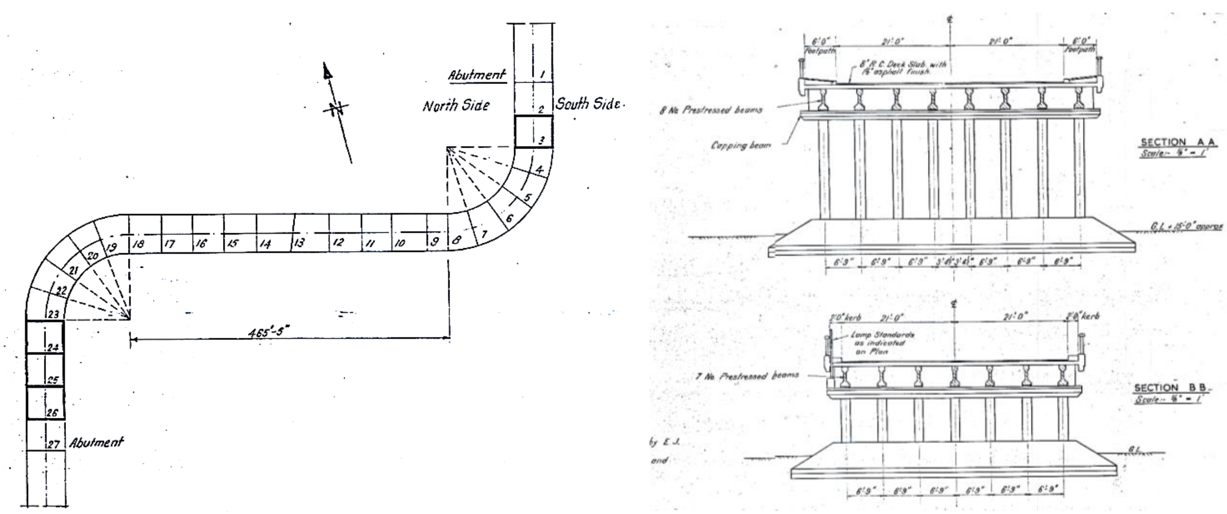

Figure 2: (LEFT) Plan of the Bridge (RIGHT) Bridge cross-sections @ Piers

A significant challenge is posed if the bridge information is insufficient for structural condition evaluation which creates significant uncertainties for the safety and serviceability of bridges. Therefore, the NEDUET research team visited the bridge site to collect the bridge information, confirm the geometry and carried out NDE tests on main structural members of the bridge. The collected bridge data was used in a developed BDMS tool by (Rafeeqi at el., 2003), to obtain the current state of the bridge based on the condition rating technique. Once the current level of the bridge and material characteristics are known, further FE Analyses is performed to predict the current capacity of the bridge against codes and propose any restrengthening and retrofitting measures required.

5. Rapid Visual Inspection & NDE Testing

The data collected from the visual inspection included the inventory record for the bridge elements and the identification of structural deficiencies based on checklists and engineering judgment. While special attention is given to areas around joints, drains, along cracks and at construction joints as these areas are often more susceptible to deterioration.

The visual inspection often provides a good overview of the condition of the structure. However, it is inefficient in case of many serious defects such as corrosion of reinforcing bars, structural cracking, pitting corrosion and others. The application of NDT in the civil infrastructure including highway bridges and pavements is to address the strength, deformability, chemical degradation, and fracture of structural materials, components, and systems. **Figure 3** shows the NDE tests executed on the case study bridge.

Figure 3: **NDE Testing on Case Study Bridge**

6. BDMS Tool Rating for Existing Condition Assessment

The above collected bridge inventory data is then used for the existing condition evaluation using developed BDMS Software at NEDUET by (Rafeeqi *et al*, 2003). The developed BDMS tool evaluates and quantifies the relative importance of various elements at risk of the bridge components taking into account different evaluation criteria, such as construction materials, structural condition of the deck, superstructure and substructures and their functional importance. The BDMS tool uses deficiency points that are given to each component of the bridge and the cumulative of all deficiency points gives the bridge rating based on maintenance and rehabilitation level.

7. Finite Element Modelling

Structural Integrity Assessment utilises field instrumentation, testing, and finite element analysis (FEA) to provide insight into structural behavior and to generate estimates of realistic load distributions in a bridge and its components. FEA define the true load carrying capacity of a structure and to diagnose potential deficiencies that cannot otherwise be identified. Detailed Structural Capacity Analysis (FE Modelling) is performed to evaluate the weakness and remaining life of the bridge.

This bridge structure is required to fulfil limit state, a condition of a structure beyond which it no longer fulfills the relevant design criteria. Limit state design requires the structure to satisfy two principal criteria: the ultimate limit state (ULS) and the serviceability limit state (SLS). ULS is not a physical situation but rather an agreed computational condition that must be fulfilled, among other additional criteria, in order to comply with the engineering demands for strength and stability under design loads. While the serviceability limit state is the design to ensure a structure is comfortable and useable. This includes vibrations and deflections (movements), as well as cracking and durability.

For the purpose of FE analysis the bridge has been modelled in (SAP2000), **Figure 4**. This program is capable of predicting large displacement behaviour of space frames under static or dynamic loading and takes into account both the geometric and material nonlinearities. In SAP2000, the deck slab has been modelled as a shell element whereas girders, diaphragms, Pier Transoms, Column Bents are modelled as line beam by defining their sections in the section designer of SAP2000. The lateral loads assigned to the bridge in terms of spectral accelerations for short period (Ss = 0.5g) and for 1.0 sec period (S_1 = 0.2g) with site class S_D. Eigen value analysis has been performed to capture the dynamic characteristics with modal combination rule SRSS (Square Root of Sum of Squares). Based on the geometric design of existing bridge, the base shear and inter story drifts are obtained through response spectrum analysis.

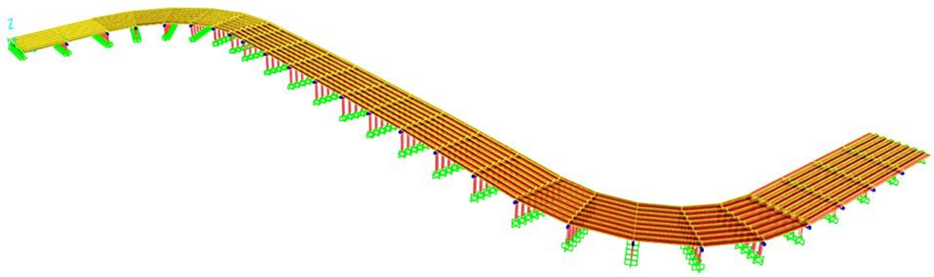

Figure 4: 3D FE Model of the Bridge

The material properties for the evaluation of the bridge are taken from the NDE tests and engineering judgement as;

- f'c = 4000 psi for Deck Slab
- f'c = 6000 psi for pre-stressed Beams and Diaphragms
- f'c = 5000 psi for pier transoms and column bents
- f'c = 5000 for foundations

The steel behaviour is represented using a uniaxial bilinear stress-strain model with kinematic strain hardening. The yield (fy) and ultimate (fu) strengths of steel bars were taken, respectively, as 60 ksi and 90 ksi whereas elastic modulus (*Es*) was taken as 29000 ksi.

7.1. Vehicular Loads

The vehicular loads given in West Pakistan Highway Code (WPHC, 1967) that consist of Truck Train Loading (Truck-A Loading) and Military 70R Tank Loading and the AASHTO Live Loads shown in **Figure 5** are employed.

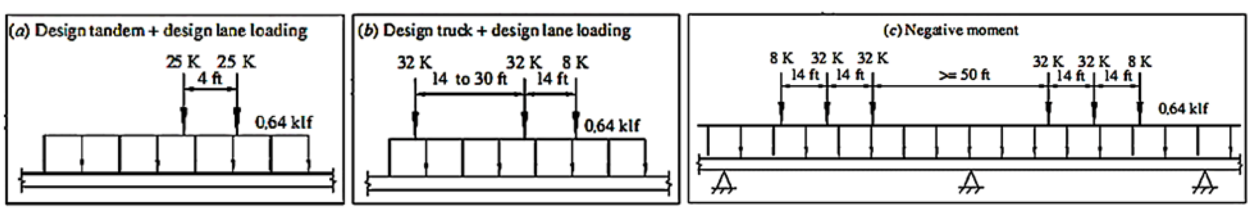

Figure 5: AASHTO HL-93 Loadings and Load Combinations (AASHTO, 2007)

7.2. Bridge Super Structure Assessment

For the assessment of deck slab, (AASHTO, 2007) method is employed, in which the deck is assumed to be a continuous beam along the girders of the bridge. While grillage approach results are used for the assessment of girders, **Figure 6**. There are three different sections of I-girders generated with the varying depths as 3'-3", 3'-6" and 4'-0", similar to that of the constructed bridge.

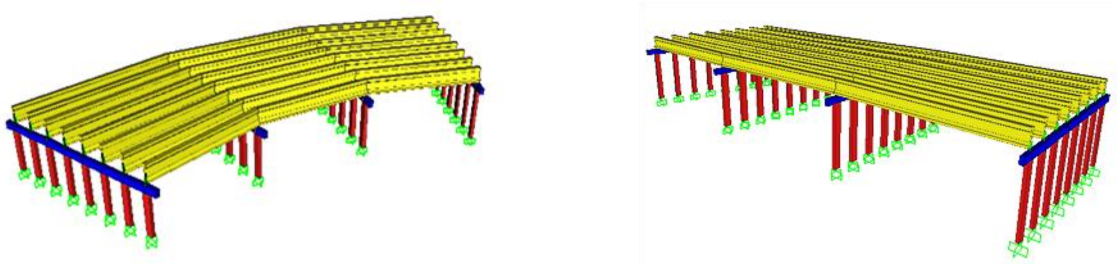

Figure 6: 3D FE Model: (LEFT) East Ramp Span (RIGHT) Main Span

7.3. Bridge Sub Structure Assessment

Demand capacity ration (DCR) is an early and much used assessment approach developed in the mid-1980s by the Applied Technology Council (ATC) and known as ATC-6-2 (ATC, 1983). Forces resulting from elastic analysis demand of a bridge structure are compared with strength (capacity) to provide a demand/ capacity ratio for actions at different parts of the structure. Assessment of the flexural and shear capacity of the RC bridge bents, **Figure 6**, has been validated against the FE analysis results that takes into account the variation of vertical loads in the bents' columns and the provided reinforcements in the structural members. This gives a satisfactory prediction of the bents' overall performance. While, piers are checked for gravity, live (vehicular) and lateral loads. Furthermore to check for lateral load capacity, multi-modal response spectrum analysis has also been performed.

8. Conclusion & Recommendations

This bridge evaluation study is intended to identify areas of potential deficiency and how to mitigate these deficiencies to the desired performance level. From observations, visually and through exploratory investigations, and from the results of the structural analyses and design reviews performed, following are the main observations;

1) Spalling of concrete in pre-stressed girders, pier column bents with exposed corroded reinforcement at some locations are observed and recommend to be repaired at once.
2) Bridge piers appears to be in good condition.
3) BDMS software shows a maintenance urgency index as G, meaning minor maintenance that requires no immediate plans for repair.
4) The strengths values are estimated based on the several NDE tests and engineering judgment are as follows;
    - f'c = 4000 psi for Deck Slab
    - f'c = 6000 psi for pre-stressed Beams and Diaphragms
    - f'c = 5000 psi for pier transoms and column bents
    - f'c = 5000 for foundations
5) The performance in both directions for piers is obtained, which shows that the capacity of the structure is adequate against the applied loads.
6) The bridge drifts and DCRs for lateral force-resisting system elements determined in accordance with the AASHTO are within the prescribed limits.
7) All columns of the bridge were found to have adequate reinforcement and load carrying capacity for the prescribed loading

The results of detailed assessment using the three Tier process show that the study bridge has adequate capacity and strength confirming to the prescribed codes and requires no immediate retrofitting interventions. Furthermore, in previous studies conducted by NED University for similar bridges in Karachi (Khan *et al.* 2013) using BDMS, the bridges falling under the category "G" have shown to have significant remaining life in the range of 10 to 15 years exceeding the design life of the bridge.

9. Acknowledgement

The authors would like to thank NED University for allowing to proceed with the project and permitting the use of the NDE testing equipment's. Special thanks to Mr. Shahid, Senior Lab Engineer at the Department of Earthquake Engineering, NED UET, and other technicians, that assisted in performing and collection of NDE test results.

10. References

AASHO, (1961). AASHO Standard Specifications for Highway Bridges, 8th Edition. USA.

AASHTO, (2006). "LFRD Bridge Design Specification", American Association of State Highway and Transportation Officials, USA.

Ali, S.M., Khan, A.N., Rahman, S., and R, A.M, (2011). "A Survey of Damages to Bridges in Pakistan after the Major Earthquake of 8 October 2005". *Earthquake Spectra,* No. 27, pp. 947-970.

ATC, (1983). "Seismic Retrofitting Guidelines for Highway Bridges", Report ATC-6-2, Applied Technology Council, USA.

Basöz, N. and Mander, J.B., (1999). Enhancement of the Highway Transportation Lifeline Module in HAZUS, Final Pre-Publication Draft (#7) prepared for the National Institute of Building Sciences.

BCP (2007). "Bridge Code of Pakistan: Seismic Provisions for Pakistan", The Ministry of Housing and Works, Government of Pakistan.

Choi, E., DesRoches, R., and Nielson, B., (2004). "Seismic fragility of typical bridges in moderate seismic zones". *Engineering Structures*. Vol. 26, pp. 187–199.

Hwang, H., Liu, J. B., and Chiu, Y.-H., (2000). "Seismic Fragility Analysis of Highway Bridges", Report No. MAEC RR-4, Center for Earthquake Research Information, USA.

Khan, R.A., Kumar, M., Ahmed, M., Rafi, M.M., Lodi, S.H., (2015). "Earthquake Damage Assessment of Bridges in Karachi", *Ned University Journal of Research - Structural Mechanics*, Vol. XII, No. 3, pp. 45-62.

Rafeeqi, S. F. A., Lodi, S. H. & Khan, R. A. (2003). "Bridge Database Management System (BDMS)", *Proceedings of 2nd International Conference on Construction in the 21st Century (CITC-II)*, Editors: S.M. Ahmed, Hong Kong Polytechnic University, HongKong, China, pp. 554-559.

SAP2000/Bridge (2009). "Bridge Analysis and Design Manual", Computer and Structures, USA.

Shinozuka, M., Feng, M. Q., Lee, J., and Nagaruma, T. (2000). "Statistical analysis of fragility curves". J. *Engrg. Mech. ASCE*, Vol. 126, No. 12, pp. 1224-1231.

WPHC 1967. Code of Practice: Highway Bridges. Government of West Pakistan.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Numerical Modelling of Three-Dimensional Flow Structure in Open Channel having Patches of Aquatic and Riparian Vegetation

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Abstract

Aquatic vegetation exists in open channels alters the flow structure and adds more roughness along its wetted perimeter. Numerical technique is used to analyze the mean flow structure in riverine structure by assuming aquatic vegetation consists of rigid staggered patches in the rectangular channel. To solve three dimensional Reynolds Averaged Navier Stokes (RANS) equations CFD package FLUENT was used in which mean flow characteristics and turbulence closure was captured well by Reynolds Stress Model (RSM). The influence of fully emerged and submerged staggered vegetation patches on flow characteristics was observed. Riparian regions having vegetation patches acquires lower flow velocities, smaller Reynolds stresses making it favorable for ecological growth and sediment deposition, whereas central portion of the channel possess higher flow velocities. Above the submerged vegetation zones, upsurge of flow velocities was observed.

Keywords

Mean Flow Structure, Numerical Simulation, Staggered Patches, Free Regions

1. Introduction

Flow characteristics in rivers are influenced by aquatic and riparian vegetations. Vegetation in open channels is the important components which significantly affect the riverine flow structure, stabilizes river channels, reduces bank erosions, improving quality of water, provides physical environment for wildlife habitat and in many other aspects which are important for the society. (Carpenter and Lodge, 1986; Sand-Jensen, 1998; Crowder and Diplas, 2002; Kemp *et al*., 2000; Tal and Paola, 2007). On the other hand, flood carrying capacity of conveyance channel is decreased by the increased resistance due to vegetation. Therefore, for proper management of the riverbanks proper balance between flood controls and riparian eco-diversity is maintained (Anjum, 2018). In river channels aquatic and riparian vegetation forms mono-specific patches which interacts with flow in very non-linear growth. These patches of vegetation are formed due to the clonal growth resulting in formation of vegetation mixture (Puijalon *et al*., 2008; Sand-Jensen and Madsen, 1992; Anjum, 2018).

Many researchers attempted to predict flow mechanics having patches of vegetation. (Maji *et al.,* 2017; Liu *et al.,* 2018*;* Wunder *et al.*, 2018; Kim *et al.*, 2019; *Yılmazer et al.*, 2019) investigated the mean flow structure and morphological changes behind the isolated vegetation patch.

However, flow structure having more than one patch is considerably different from single patch, (Meire *et al.*, 2014) experimentally studied the interaction and effects of two adjacent vegetation patches on flow features. Many researchers adopted numerical simulation, CFD techniques for investigation of mean flow structure and interaction of emerged, submerged and layered patches in open channel flows (Anjum *et al*., 2018; Anjum and Tanaka, 2019a; Ghani *et al*., 2019a; Anjum and Tanaka, 2019b; Ghani *et al*., 2019b). (Zhao and Huai, 2016) Investigated the flow through discontinuous submerged vegetation patches and they employed LES model for turbulence closure. Recently (Ghani *et al*., 2019a) investigated the flow structure through circular staggered vegetation patches. Structure and configuration of vegetation patches are important factors which effects vegetation species to alter flow structure and sedimentation processes (Licci *et al*., 2019).

Combined effect of aquatic as well as riparian vegetation patches still needs to be explored. Recent study focuses on the investigation of flow structure and the mean flow characteristics due to mutual interference of riparian(outer) and aquatic(central) staggered patches in the free regions behind the vegetation patches. Reynolds stress turbulence model (RSM) is applied to investigate the flow characteristics. 3D numerical code CFD-FLUENT is used to solve Reynolds Averaged Navier Stokes equation.

2 Materials and Methods:

2.1 Governing Equations:

In an open channel when flow was fully developed numerical model was set up by employing Reynolds Averaged Navier-stokes equations for continuity and momentum. Mathematically these equations can be written as.

$$\frac{\partial U_i}{\partial X_i} = 0 \qquad (1)$$

$$U_j \frac{\partial}{\partial X_j}(U_i) = \frac{V\,\partial}{\rho \partial X_i}\left(\frac{\partial U_i}{\partial X_i} + \frac{\partial U_j}{\partial X_i}\right) - \frac{1}{\rho}\frac{\partial P}{\partial Xi} + (-\rho \overline{u}_i \overline{u}_j\ ) \qquad (2)$$

Where,

U_i = Velocity component in x_i direction, U_j = Velocity component in x_j direction,

V = Kinematic Velocity, ρ = Water Density, P = Pressure and $\rho \overline{u}_i \overline{u}_j$ = Reynolds stresses

In generalized form transport of Reynolds stresses can be written as (Versteeg and Malalasekera, 1995),

$$\frac{\partial R_{ij}}{\partial t} + C_{ij} = P_{ij} + D_{ij} - \varepsilon_{ij} + \prod\nolimits_{ij} + \Omega_{ij} \qquad (3)$$

Rate of Change of Reynolds Stresses	+	Transport of Reynolds stresses by Convection	=	Rate of production Reynolds stresses	+	Transport of Reynolds stresses by diffusion	-	Rate of Dissipation of Stresses
			+	Transport of Stresses Due to turbulent Pressure-strain interactions	+	Transport of Stresses due to rotation		

2.2 Vegetation Modelling:

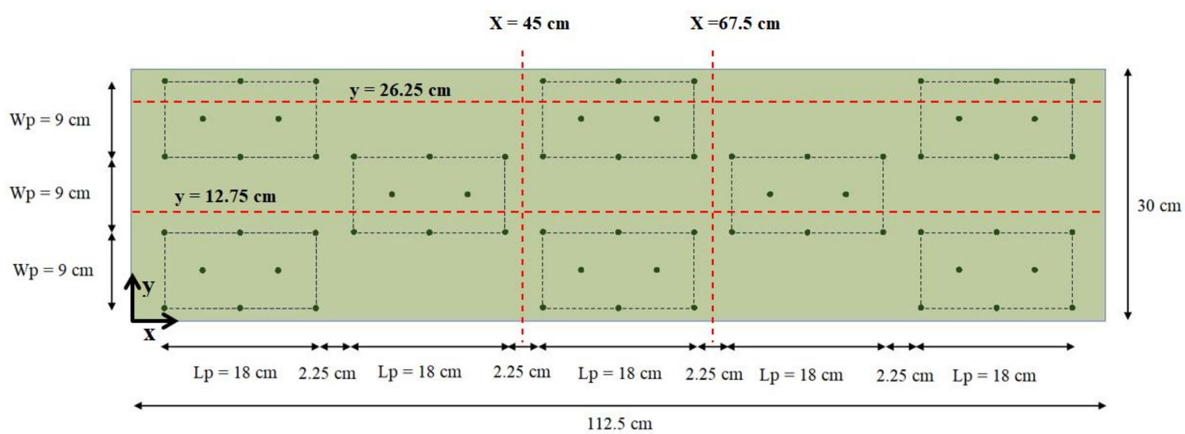
2. 2.1 Model Setup and Boundary Conditions

In this study computational flow domain of 1.12m long and 0.3 wide was selected, and Ansys Workbench was used for modelling the geometry. Stream wise, lateral and depth wise directions are represented along x, y and z axis respectively. Rectangular Staggered emerged and submerged vegetation patches are modelled with the rigid cylinders of 7.5mm diameter. The spacing within the patch kept same (i-e S/d = 6 or S/d = 6 where S is the spacing between two adjacent cylinder and d is the diameter of vegetation cylinder). 2 cases depending upon the height of vegetation patches i-e totally emerged and totally submerged were considered for the present study. The modelled sketch of Case A is shown in Fig. 1a whereas Fig. 1b shows the specified locations where various flow characteristics were computed. Hydraulic conditions used in this numerical simulation is listed in Table 1.

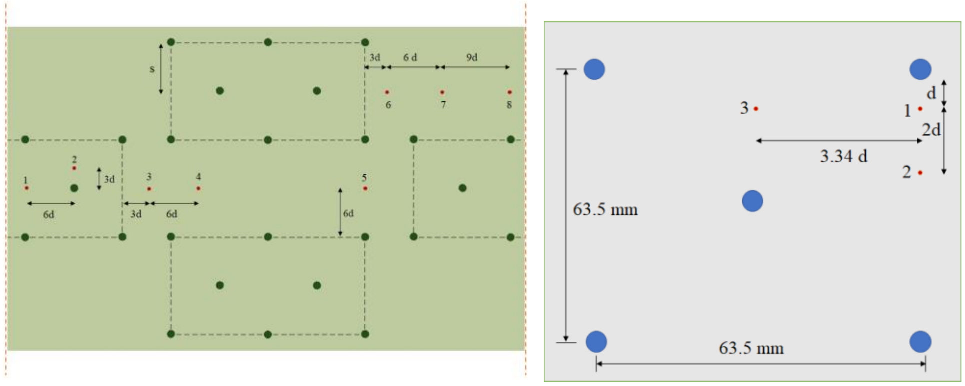
Table 1: Hydraulic conditions of numerical simulation

Cases	Vegetation Height (cm)	Submergence Condition	Spacing (S/d)	Depth z (cm)	Discharge (l/s)	Velocity U (m/s)
A	12	Emerged	6	9.7	11.4	0.392
B	6	Submerged	6	9.7	11.4	0.392

Unstructured mesh grid with tetrahedral elements were adopted for meshing the flow domain. Initially the mesh grid adopted have 1.4 million grid points. However, to check the accuracy of computed results mesh independence test has been performed. For mesh independence nodes were doubled in the transverse and lateral direction giving 1.9 million grid points and the difference between the simulated velocities of two meshes were less than 1%. The skewness of final mesh adopted was checked and found that mesh generated is of high quality and would not affect the solution accuracy.



(a)



(b) (c)

Figure 1: (a) Schematic Diagram of flow domain, (b) Critical locations numerically simulated Cases A and B, (c) Critical points where (Liu *et al*., 2008) experimentally determined velocities.

Inlet/Outlet was mapped with periodic boundary condition to ensure uniformity in the computational flow domain. Velocity component at walls, bed and walls of vegetation element should be zero so they were specified as no-slip boundary conditions. Water surface was deal with rigid lid assumption and plane of symmetry boundary condition was provided. The generated vegetation model was imported to CFD package FLUENT for numerical simulation and for turbulence closure Reynolds Stress Model (RSM) was employed. For numerical setup SIMPLE scheme was used for pressures coupling velocity, standard, least squared cell based and 2nd Order upwind was used for spatial discretization. All the residual plots were converged after reaching $1 \text{ x } 10^{-7}$.

2.2.2 Numerical Model Validation:

Experimental data of (Liu *et al*., 2008) was used for the validation of vegetation model. (Liu *et al*., 2008) performed experiments in flume of length 4.3m, width of 0.3m and slope of flow domain was set to .0003. Acrylic cylinder of 6.35 mm was used to model vegetation of height 76 mm. To avoid large mesh structure and computational time only 1.2 m long flow domain was model.

Table 2: Experimental Conditions (Liu et al., 2008)

Experiment	Spacing	Submergence Condition	Flow Rate (l/s)	Depth (cm)
2.1	5s	Emergent	4.4	6.5
3.1	5s	Submergent	11.4	11.4

Experiment 2.1 and Experiment 3.1 was replicated to validate numerical model for fully emerged and submerged vegetation cases. Numerically simulated velocities were computed at the locations Figure 1c specified by (Liu *et al*., 2008). Figure 2(a-b), shows the comparison that numerically computed velocities are very close to experimentally measured velocities, which indicates vegetation model is validated.

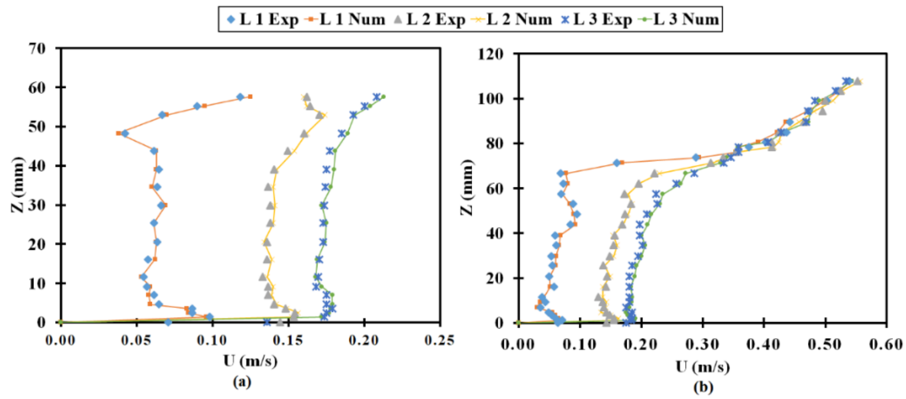


Figure 2: Comparison of computed and experimental mean velocity profiles, (a) Exp 2.1, (b) Exp 3.1

3. Results and Discussion:

3.1 Mean Flow Characteristics:

Mean streamwise velocity profiles at various specified locations were computed and normalized it with average sectional velocity *U* shown in Figure 3 (a-b). For emerged vegetation patches, normalized mean streamwise velocity *u/U* remains almost constant up to z/h<0.75, however velocity dip is observed in the region z/h>0.75. Within the central patch at location 1 upstream of dowel bar, flow is retarded due to drag offered by the vegetation cylinder. However, flow is accelerated towards the side of cylinder at location 2. At the downstream of central and outer vegetation patches, simulated normalized longitudinal velocities were reduced locally due to sheltering effect. Outer vegetation patches offered more resistance to flow, velocities computed downstream of outer vegetation patch is 8.7 % less as compared to central patch.

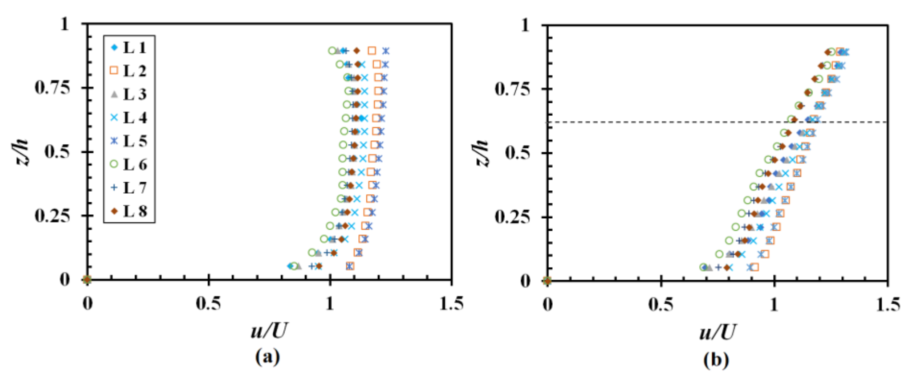


Figure 3: Vertical profiles of normalized mean streamwise velocities, (a) Case A, (b) Case B. Dotted line in (b) represent the top of submerged vegetation.

Free regions behind the outer and central vegetation patch due to sheltering effect velocities were reduced, results remained consistent with (Naveed *et al*., 2018; Zhao and Huai, 2016), However, at location 5 upstream of central vegetation patch highest magnitude of mean velocity was observed due to acceleration of flow. Overall increased magnitude of velocities was observed in the central portion of channel. For submerged vegetation patches, magnitudes of mean velocity increases towards the free surface and follows the velocity log law. (Naveed *et al*., 2018; Zhao and Huai, 2016) found that within patch region velocities are more as compared to free regions.

However, flow comprising of staggered vegetation patches velocities in the patch region in comparison to free region is more up to region h/z<0.62 (i-e within the submerged vegetation patch).

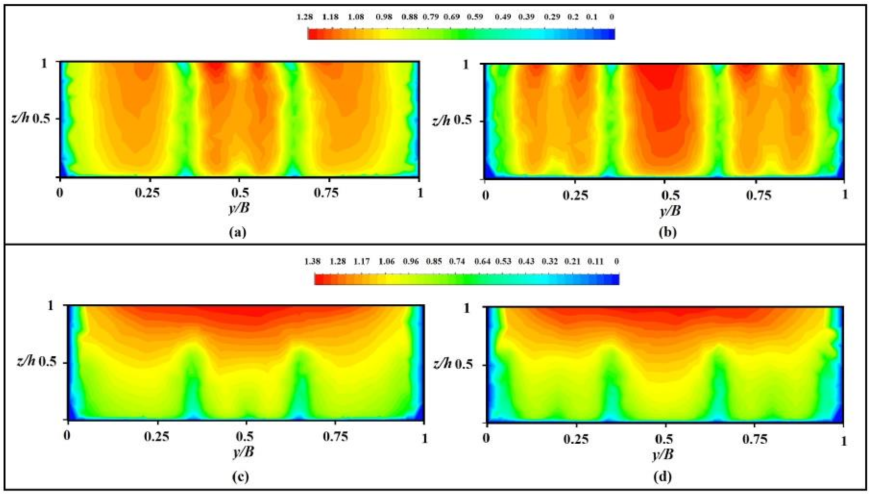


Figure 4: Mean stream wise velocities *u/U*, for case A at cross section (a) X = 45 cm and (b) X = 67.5 cm, for Case B (c) X = 45 cm and (d) X = 67.5 cm respectively.

Contour plots of simulated normalized longitudinal velocities at cross-section was determined x=45 cm (upstream of outer and downstream of central patch) and x = 67.5 cm (downstream of outer and upstream of central patch). Contour plots shows non-uniform distribution of mean velocities at all section Fig. 4(a-d). It can also be seen that velocity magnitude near side walls and bed of channel is minimum. For both Cases A and B common region located between outer and central vegetation have also minimum values of velocity due to the formation of shear layer. Central portion of the channel have higher magnitude of velocity as shown in Figure 4(a-b) which are consistent with results obtained in Location 4,5 and 6. In submerged vegetation patches, increment in values of velocities were observed above z/h>0.5. Possible upsurge of flow velocities is due to the momentum exchange between overlying flow and vegetation (Ghisalberti and Nepf., 2005). However lower velocities in the vegetated reason is observed in Case B due to large generated turbulence.

3.2 Turbulence Characteristics:

Turbulence characteristics can be represented by the Reynolds shear stresees(Zhao and Huai, 2016). The development of the turbulent coherent structures in open channels is due to the evolution of Reynolds shear stresses. (Okamoto and Nezu, 2013). Figure 5(a-b) and Figure 6(a-b) shows the contour plots of Spanwise shear stresses normalized with U^2 computed at y = 12.75 cm and 26.25 cm. u' and v' represents the variation in the streamwise and transverse directions respectively. From Figure 5a, at y = 12.75 cm Reynolds shear stresses remains stable throughout however within the vegetation patch slightly higher magnitude of stresses are observed. It can also be observed that vegetation patch near the bed very low or negative magnitude of Reynolds shear stresses can be witnessed. From Figure 5b, almost zero or negative values of shear stresses are observed throughout the depth. The magnitude of stresses are slightly increased at the tail end of the patch region, wake region behind patch and at the top of free surface. The fluctuations of reynolds shear stresses near the top is due to the formation of Kelvin–Helmholtz eddies (Zhao and Huai, 2016).

For case B large fluctuations in the transverse Reynolds shear stresses are observed. At y = 12.25 cm near the bed magnitude of Reynolds stresses are almost zero or negative near the bed, whereas Reynolds shear stress remain invariant up to vegetation height and shows fluctuations above vegetation top is due to upsurge of velocities and exchange of momentum between vegetation and layers of overflow. Overall magnitude of Reynolds shear stresses for both cases at y= 26.25 cm is very much smaller as compared to y =12.5 cm. Magnitude of Reynolds shear in the outer region flow is almost 90 % small as compared to the central portion. Fluctuations of Reynolds Shear stresses in both cases significantly effects the dynamics of sediments which influences the particle size distribution carried by the water.

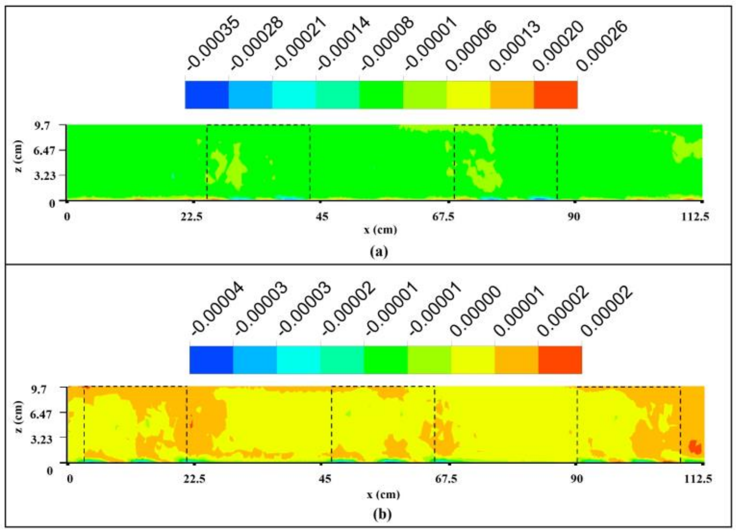


Figure 5: For Case A contour diagrams of Reynolds stresses (-*u'v'/U*^{2}), (a) y = 12.75 cm, (b) y = 26.25 cm.

Outer vegetation patch and free regions acquire very small Reynolds shear stresses making it favourable for the deposition of sediments and provides environment for the growth and nourishment aquatic habitat. Wheras due to higher flow velocities and Reynolds shear stresses in central portion a small deposit will take place. Concluding that riparian vegetation patches alters the flow structure in that region and protects the banks from erosions, However the aquatic vegetation patches in the central portion of open channels increases the flow velocities.

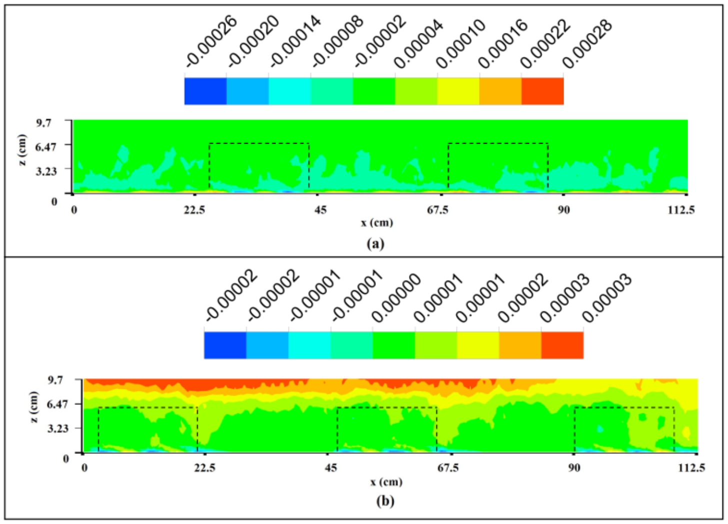


Figure 6: For Case B contour diagrams of Reynolds stresses (-*u'v'*/U^2) (a) y = 12.75 cm, (b) y = 26.25 cm.

Conclusion:

Computational fluid dynamics technique used for the investigation of mean flow structure through staggered vegetation patches. For solving RANS equations FLUENT code was used and Reynolds Stress model (RSM) employed for turbulence closure. Conclusions from this study are as follows:

- In the presence of staggered vegetation patches, flow velocities in the free regions located behind vegetation patches are reduced due to sheltering effect. Magnitude of flow velocities in the central portion of channel is high as compared to outer portions. Velocities computed downstream of outer vegetation patch is 8.7 % less as compared to central patch.
- In submerged vegetation patches, increment in values of velocities were observed above z/h>0.5 due to the exchange of momentum between cylinders and overlying flows.
- Riparian vegetation patches protects river banks making vegetated and free regions favourable for the deposition of sediments and provides environment for the growth and nourishment of aquatic habitat. Whereas aquatic patches accelerates the open channel flows.

Acknowledgement:

Authors acknowledge the contribution of Higher Education Commission Pakistan for providing CFD Software service at University of Engineering and Technology, Taxila.

References:

Anjum, N., (2018) “Study of discontinued layered vegetated open channel flow”, M.Sc. Thesis, *University of Engineering and Technology.*

Anjum, N., Ghani, U., Pasha, G. A., Latif, A., Sultan, T., and Ali, S., (2018). “To investigate the flow structure of discontinuous vegetation patches of two vertically different layers in an open channel”, *water*, Vol. 10, No. 1.

Anjum, N., and Tanaka, N. (2019a). "Study on the flow structure around discontinued vertically layered vegetation in an open channel", *Journal of Hydrodynamics*, https://doi.org/10.1007/s42241-019-0040-2.

Anjum, N., and Tanaka, N. (2019b). "Hydrodynamics of longitudinally discontinuous, vertically double layered and partially covered rigid vegetation patches in open channel flow", *River Research and Applications,* https://doi.org/10.1002/rra.3546

Carpenter, S, R., and Lodge DM (1986). "Effects of submersed macrophytes on ecosystem processes", *Aquatic Botany*, Vol. 26, pp. 341-370.

Crowder, D, W., and Diplas, P., (2002). "Vorticity and circulation: spatial metrics for evaluating flow complexity in stream habitats", *Can J Fish Aquat Sci*, Vol. 59, No. 4, pp. 633–645.

Ghani, U., Anjum, N., Pasha, G. A., & Ahmad, M. (2019a). "Investigating the turbulent flow characteristics in an open channel with staggered vegetation patches*", River Research and Applications*, Vol. 35, No. 7, pp. 966–978.

Ghani, U., Anjum, N., Pasha, G. A., & Ahmad, M. (2019b). "Numerical investigation of the flow characteristics through discontinuous and layered vegetation patches of finite width in an open channel*", Environmental Fluid Mechanics,* Vol. 6, No. 6, pp. 1469-1495.

Ghisalberti, M., and Nepf, H., (2005). "Mass transfer in vegetated shear flows", *Environmental Fluid Mechanics,* Vol. 5, No. 6, pp. 527–551.

Kemp, J, L., Harper, D, M., and Crosa, G, A., (2000). "The habitat-scale eco-hydraulics of rivers", *Ecol Eng*, Vol. 16, No. 1, pp. 17–29.

Kim, S, H., Kimura, I., and Shimizu, Y., (2019). "Experiment and Computation of Morphological Response to a Vegetation Patch in Open-Channel Flows with Erodible Banks", *Water, Vol.11, No. 11*.

Licci, S., Nepf, H., Delolme, Delolome, S., Marimonier, P., Bouma,T, J., Puijalon, S., (2019). "The role of patch size in ecosystem engineering capacity: a case study of aquatic vegetation", *Aquatic Sciences*, https://doi.org/10.1007/s00027-019-0635-2

Liu, C., Hu, Z., Lei, J., and Nepf, H., (2018). "Vortex Structure and Sediment Deposition in the Wake behind a Finite Patch of Model Submerged Vegetation", *Journal of Hydraulic Engineering,* Vol. 44, No. 2, DOI: 10.1061/(ASCE)HY.1943-7900.0001408.

Liu, D., Diplas, P., Fairbanks, J, D., and Hodges, C, C., (2008). "An experimental study of flow through rigid vegetation", *Journal Of Geophysical Research Earth Surface,* Vol. 113, No. F4.

Maji, *B.*, Pal, D., Hanmaiahgari, Gupta, U, P., (2017). "Hydrodynamics and turbulence in emergent and sparsely vegetated open channel flow", *Environ Fluid Mech,* Vol. 17, No. 4, pp. 853-877.

Meire, D. W. S. A., Kondziolka, J. M., & Nepf, H. M. (2014). "Interaction between neighboring vegetation patches: Impact on flow and deposition", *Water Resources Research*, Vol. 50, No. 4 pp 3809–3825.

Okamoto, T., Nezu, I., (2013). "Spatial evolution of coherent motions in finite-length vegetation patch flow". *Environmental Fluid Mechanics,* Vol. 13, No.5, pp. 417– 434.

Pal, D., Halder, B., Hanmaiahgari, P, R. (2017). "Comparison of Turbulent Hydrodynamics with and without Emergent and Sparse Vegetation Patch in Free Surface Flow", *In: Garg V., Singh V., Raj V. (eds) Development of Water Resources in India. Water Science and Technology Library, vol 75. Springer*

Puijalon, S., Bouma, T, J., Groenendael, J, V., and Bornette, G., (2008). "Clonal Plasticity of Aquatic Plant Species Submitted to Mechanical Stress: Escape versus Resistance Strategy", *Annals of Botany*, Vol. 102, No. 6, pp. 989–996.

Sand-Jensen, K., Madsen, T, V,. (1992). "Patch dynamics of the stream macrophyte, *Callitriche cophocarpa", Freshwater Biololgy*, Vol. 27, No. 2, pp. 277-282.

Sand-Jensen, K., and Pedersen, M, L., (2008). “Streamlining of plant patches in streams”, *Freshwater Biology*, Vol. 53, No. 4, pp. 714-726.

Tal, M., and Paola, C., (2007). “Dynamic single-thread channels maintained by the interaction of flow and vegetation”, *Geology*, Vol. 35, No. 4, pp. 347–350.

Versteeg, H. K. and Malalasekera, W. (1995). *An introduction to computational fluid dynamics, The finite volume method*, Longman Scientific & Technical, Longman Group Limited, England, pp. 75.

Wunder, S., Trevisson, M., Heckele, C., Chagot, L., Murphy, B., McLelland, S., Moulin, F., Eiff, O., (2018). “Near wake of emergent vegetation patches in shallow flow”, *River Flow 2018 - Ninth International Conference on Fluvial Hydraulics*, Vol. 40, https://doi.org/10.1051/e3sconf/20184002057

Yılmazer, D., Ozan A, U., and Cihan, K., (2019). “Flow Characteristics in the Wake Region of a Finite-Length Vegetation Patch in a Partly Vegetated Channel”, Water Vol. 10, No.4.

Zhao, F., Huai, W., (2016). “Hydrodynamics of discontinuous rigid submerged vegetation patches in open-channel flow”, *Journal of Hydro-environment Research*, Vol. 12, pp.148-160

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Experimental Investigation on Different Geometrical Shapes of Dual Bridge Pier Scouring

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Abstract

Bridge piers play an extensive role in transport network of a country; enough investigation ought to be adopted in bridge piers design. Scouring is the most critical factor that compels to adopt optimum measurements and shapes of the piers. Without prescribed investigation, it turns out to be remarkably difficult to counter and diminish the impacts of scouring—which at last lead to pier failure. In this study, different trials were accomplished in order to achieve an optimized shape of piers— for scour depth—and their enhancement in lateral direction for double, circular, diamond and square shaped pier. The investigation shows that square piers have prominent scour depth as compared to circular and diamond shaped bridge piers. It also indicated that in case of circular piers, significant reduction, up to 45%, in the scouring depth takes place on upstream side of the pier. When these results were compared with square shaped piers—in this case 35% reduction takes place in scouring depth as compared to diamond shaped piers. A series of experiments were conducted at three different discharges and constant flow depth under clear water conditions on uniform sediment size. Scour depths regarding additionally exhibited in this study, with contour maps.

Key Words

Local scour, Maximum scour depth, Clear-water condition, flow intensity & pier shapes

1. Introduction

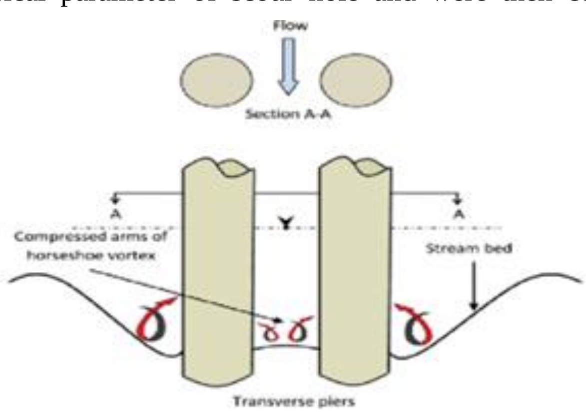
Scouring around a bridge pier exposes the surface of pier—that is supposed to be covered by stream bed —and is considered as the leading and extensive cause of bridge failure, bringing about extremely high reproduction cost. It may cause a substantial loss of human lives, while over forecast may bring about uneconomical structure. Subsequently, it is necessary to, for sustainable structure, estimate the scour depth (FDOT, 2005).

Even though a pier is a crucial and key part of a bridge, however, it acts as an obstacle to the flow of water and, accordingly, leads towards scouring (Khan et al., 2017). According to the study (Yanmaz, A.M.; Üstün, 2001) the complexity of pier scour modeling has no single method that overlays all the constraints of flow, sediment, river, and pier characteristics universally. Due to this reason, very limited thorough mathematical methods for predicting scour depth exist.

The actual values of pier scour depth could be obtained from field measurements, but it is very difficult, especially during flood storms. Therefore, to understand the scouring phenomena, it is essential to measure the depth of the small-scale pier scouring. In this case, the researchers (Khan et al., 2017) conducted a number of experiments to study the depth of the pier travel and its variation with independent variables. Based experimental work, Melville and Chiew (Melville and Chiew., 1999) reported the temporal variation of depth of the pier scouring. Experimental results indicated that the scouring depth increases asymptotically as there is increase in time. The results also indicated the variation of the pier perforation depth with independent non-dimensional input variables, including the intensity of the flow, the ratio between the thickness of the sediment and the depth of the flow (Ettema, R. 1980) explained the variation of the depth of the pier scouring with the increase in roughness of the sediments through a series of experiments. It was mentioned that "the scouring process has changed abruptly with the change in the gradation of the sediment while scouring with clear water due to the following two key reasons.

1. The thicker material filled the grooves on the surface of the scour hole reduces the downward impact of the flow thereby reduces the effect of scouring.
2. Extra down flow energy is dissipated by the gaps and spaces between the coarser sediments, thus reducing the depth of the scouring.

In addition to that (Sheppard et al. 2004) analyzed the influence of scour load on pier scour depth through a series of experiments in a large flume at the USGS research center. Similarly, another attempt of experimental investigation was conducted on semi-integral bridge piers for prediction of time-dependent local scour for both coarse and fine sediments (Piers et al., 2013). Their study revealed that pier scour depth increases with an increase in time and discharge but decreases with increasing sediment size. The study was focalized on time, discharge, and sediment size; however, it was not aimed at analyzing effects of variation in pier sizes and shapes. Furthermore, it was concluded that increase in the pier size and flow intensity increases the pier scour depth, plus the scour hole surface area and volume (Ozalao et al., 2016). The bridge piers with 100 inclination were found more effective against scouring as compared to piers inclined at 150. The research (Khwairakpam et al., 2012) analyzed the characteristics of scour hole around a vertical pier under clear water scouring conditions. It was primarily analyzed that the scour hole characteristics depend mainly on Froude number and flow depth. Following that, empirical equations were derived for each geometrical parameter of scour hole and were then compared with previous



experimental results and were found very close.

Figure 1: Principal Components of Local Scour Depth on Bridge Pier.

To represent the local scouring mechanism, different components of the local scouring process have been reported in clear water conditions for a constant flow depth at different flows, as shown in Figure 1.

The depth of local scouring around the pier depends on different variables, including fluid properties, bed flow and geometry of sediments and sources. The objectives of this study were: (1) to experimentally measure the depth of the scour on three different shapes of piers (i.e. dual circular, diamond and square piers) commonly used in bridge construction, (2) to study the variation of the depth of the scouring around a pier corresponding to the variation of different independent variables which includes the flow parameters (depth and flow), the geometry of the pier and the size of the sediments (3). The study of the effects of the flow and sediment parameters along with pier geometry, on the depth of the scouring around the pier, is of prime importance.

2. Experimental work

2.1 Channel description

The channel used in the experimental tests is shown in Figure 2. All experiments were conducted in the Hydraulic laboratory of Faculty of Civil Engineering at University of Engineering and Technology Taxila. The test channel was 20 m long, 0.96 m wide and 0.75 m deep, provided with glass side walls and concrete bottom. The piers were placed at 3.5 m downstream of the entrance of the channel to generate the complex flows. The channel bed was horizontally levelled prior to start of experimental work. The channel was carefully supplied with tap water at depth of 15cm and any disturbance to the upstream bed by the approaching flow was avoided. The water level in the channel was controlled by an adjustable tailgate. A vertical point gauge with 0.1mm precision on the Vernier scale was used to measure the scour depth and water level. The uniform, cohesion less bed material in the channel was 15cm thick in the scouring test section. The specific weight 26.5kN/m^3, the ratio of $D/d50 \geq 50$ is satisfied when d50 is $\geq$ 0.6mm, and this value avoids the influence of the sediment coarseness (Melville and Raudkivi 1977; Chiew and Melville 1987; Melville and Sutherland 1988). The geometric standard deviation of sediment particles ($\sigma_g = d85/d50$) was <1.3 (Raudkivi 1986; Chiew and Melville 1987) to mitigate the effect of the non-uniformity of the sediment on the scour hole. Moreover, the diameter of bed particles was 0.57mm to prevent ripple formation (Ettema 1980; Raudkivi and Ettema1977; Lanc ¸a, Simarro, and Fael 2015).

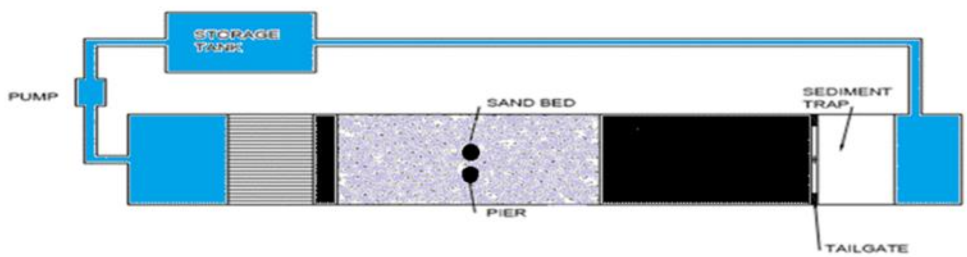


Figure 2: Open Channel in the Hydraulic Laboratory Where Experiments Were Conducted On Different Pier Shapes

2.2 Experimental Procedure

Prior to the experiment, the channel was cleaned, and the test section was filled with medium sized sediments of d50 = 0.57 mm, for all experiments. The sand was properly leveled in order to ensure the uniform water level on sand bed in the test section to fill the air spaces with water. As the channel bed got saturation point, the piers were installed at the center of the channel section. The initial bed level was used as a reference for all scour depth measurements. Water was allowed to flow over the test section and scour started around the pier. It was noticed that the scouring was initiated on the upstream face of the piers that went to spread along the sides of the pier. In the start, the depth of the flow was measured using the point gauge, flow with rectangular notch and downtime Watch. After running the experiment for the specified time 3.0 hrs, the water was drained and the scouring depths at upstream, downstream and both the sides of the basin were measured using a point gauge. The scour dimensions of the hole (length and width) were also measured and recorded. The sediment bed was again leveled, and the experiments were repeated for a new leveled bed.

Figure 3: (A) Circular Shaped Bridge Pier (B) Diamond Shaped Bridge Pier (C) Square Shaped Bridge Pier

3. Result And Discussion

3.1 Variation of Scour Depth with velocity

The pier scour depth variance is due to the variability of flow velocity for all shapes (i.e. circular, diamond and square pier models) over time intervals of 3h. It depicts that an increase in velocity increases the pier scour depth, but after a while it tends to decrease. The reason is that when the flow velocity exceeds the critical sediment velocity, it brings sediment from upstream side to the scour hole, which results in a reduction of the scouring depth.

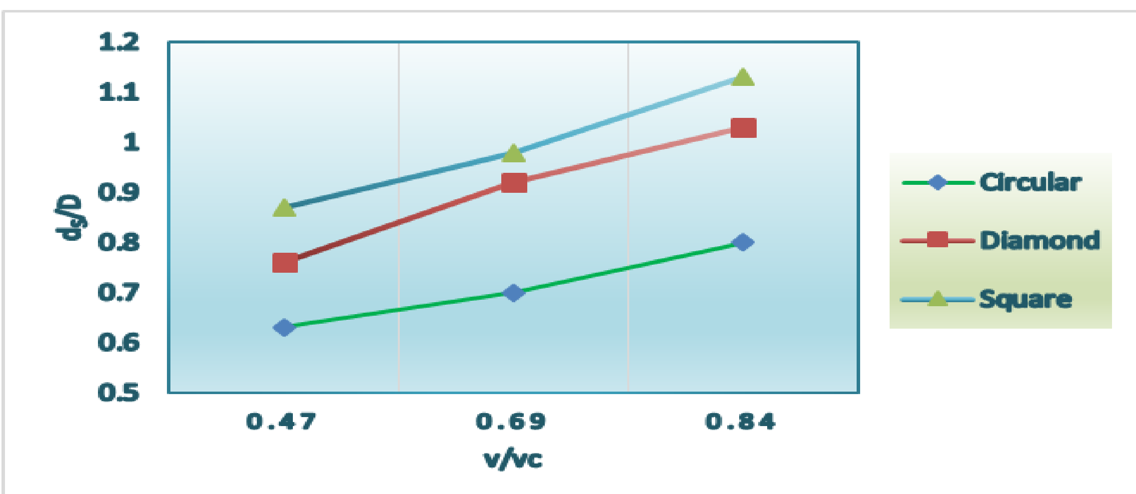


Fig 4: Variation with the Flow Velocity for Circular, Diamond and Square Pier Models for Time Interval of 3h.

3.2 Variation of Scour Depth with Discharge

The plot of pier scour depth versus discharge shows that using both the pier models, the pier scour depth increases with the increase in discharge and vice versa. It is since due to the increase in discharge that both the flow depth and velocity increases, which in turn cause an increase pier scour depth. This trend is shown in Fig. for circular, diamond and square pier models for time interval of 3h, respectively.

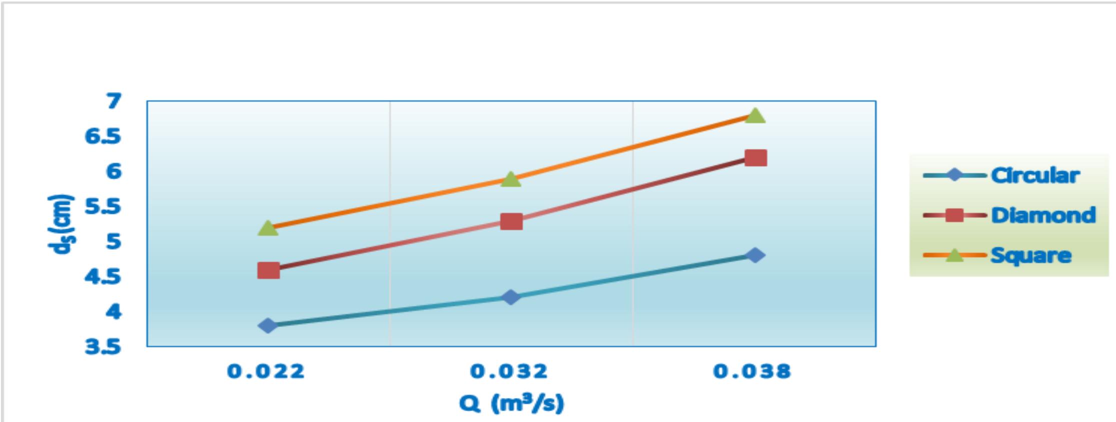


Figure 5: Scour Depth Variation with Variation in Discharge for Circular, Diamond and Square Pier Models.

3.4 Contour Maps for Scour Hole of Circular, Diamond and Square Pier Models

Figure a–c respectively presents the final scour profile in terms of contour maps, measured at the end of each experiment (after 3h). These contour plots were generated from the measurements of vertical elevations using a point gauge. It was observed from the figures that maximum scour depth was at the upstream as Square> Diamond> Circular of the pier models. The contour maps for the scour depth (and its scour hole) not only provide information about the scour process, the dimensions of the scour hole (length and width) and the topography of scour, but can also be useful in planning and designing counter measures for protection against scouring under different flow conditions and uniform sediment. In this regard, a very limited literature is available and one of the objectives of this research is to use the experimental data collected in the development of these topographic contour maps for the depth of the course of the pier. Therefore, experimental analysis data was collected so that it could be used to draw contour maps of the scour hole around the pier model. The contour maps were drawn only for the region of the scour hole using the experimental data collected in this survey. The eight points selected around the pier were to record to the measurements and to draw contour maps from points of maximum and minimum scouring depths around a pier. Figures show the contour map for the circular, diamond and square pier models, respectively, using the data obtained from the experiments. It was evident that the scour depth as well as scour hole dimensions for square pier model were large than the scour depth for circular and diamond pier model. Therefore, larger area around the pier could be affected by scouring due to square pier model as compared to the circular and diamond pier.

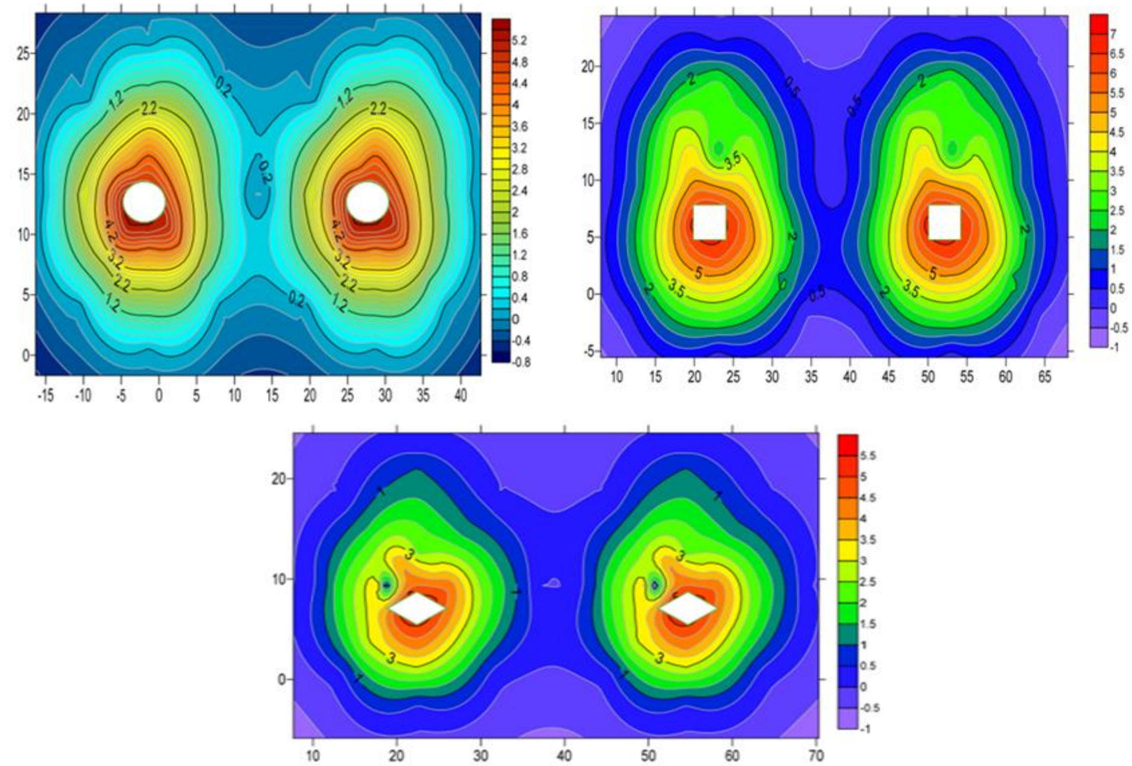


Figure 6: Contour Map for (A) Circular Shaped Pier Model (B) Diamond Shaped Pier Model (C) Square Shaped Pier Model

3.5. Comparison of Experimental Results

At the end, the experimental results obtained from this study, were compared with the results obtained by using different shapes of piers. It is evident, from the results, that circular pier faces a minimum scour depth for all flow conditions as compared to diamond and square pier models.

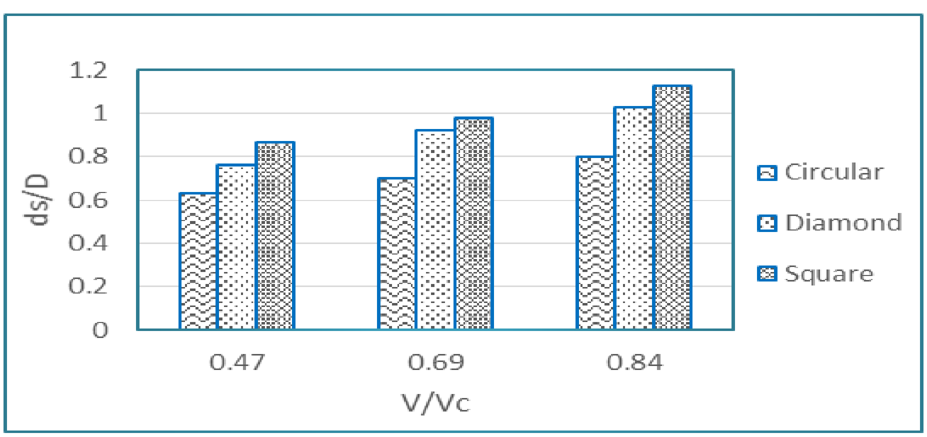


Figure 7: Scour Depth Comparison with Variation of Velocities for Circular, Diamond and Square Pier Models.

3.5 Reduction of pier scour depth

The investigation shows that square pier models have more prominent scour depth as compared to circular and diamond shaped bridge piers; showed valuable results as the circular pier significantly reduce the scouring depth at upstream side of the pier up to 45 % when compared with square and 35% when compared with diamond shaped pier.

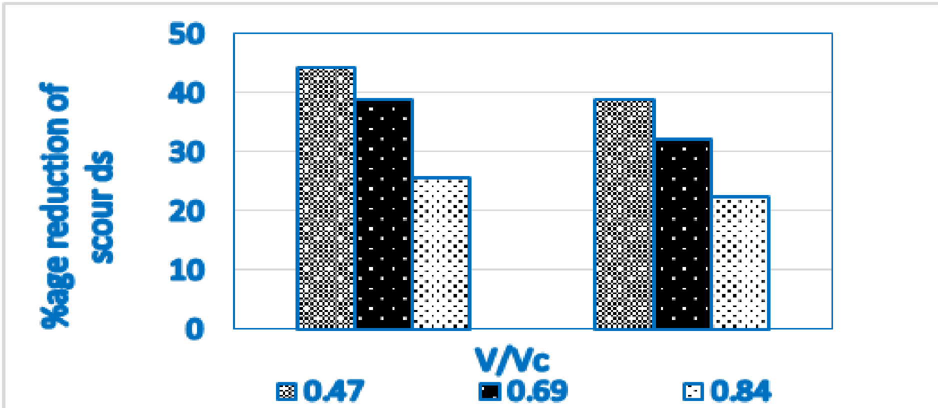


Figure 8. Percentage Reduction of Scour Depth % Versus V/ Vc Of Different Shapes.

4. Conclusions

This experimental study was conducted to assess the most efficient and an optimum pier shape in terms of reducing scour depth. It also presents the capability of various pier modifications to minimize the erosive power of flow, acting on the riverbed.

- The scour depth around a square shaped pier is greater than the scour depth around a circular and diamond one models, at all the three flow conditions.
- Flow velocity exhibits a direct relation with scour depth, around a pier. Similarly, an increase in flow velocity increases the scour depth because of its erosive action until it reaches to the critical velocity which tends to bring sediments to the scour hole from upstream, causing reduction in pier scour depth. The contour maps drawn under different conditions of flow, sediments, and pier geometry effectively reflect and exhibit the extent of scouring process—that also represents the degree of remedial measures.

5. References

FDOT. (2005). *State of Florida Department of Transportation Bridge Scour Manual*. *May*.

For, C., At, C., By, I., Melville, B. W., & Chiew, Y. (1999). *Cale for*. *May 2010*, 59–65.

Khan, M., Tufail, M., Ajmal, M., Ul, Z., & Kim, T. (2017). Experimental Analysis of the Scour Pattern Modeling of Scour Depth Around Bridge Piers. *Arabian Journal for Science and Engineering*. https://doi.org/10.1007/s13369-017-2599-7

Khwairakpam, P., Ray, S. S., Das, S., Das, R., & Mazumdar, A. (2012). *SCOUR HOLE CHARACTERISTICS AROUND A VERTICAL PIER*. *7*(6), 649–654.

Paper, T. (2016). *Experimental investigation of local scour around circular bridge piers under steady state flow conditions*. *58*(3).

Piers, I. B., Akib, S., & Rahman, S. (2013). *Time Development of Local Scour around Semi*. *7*(7), 243–248.

Yanmaz, A.M.; Üstün, ˙ I.: Generalized reliability model for local scour around bridge piers of various shapes. Turk. J. Eng. Environ. Sci. **25**, 687–698 (2001).

Ettema, R.: Scour at bridge piers. Report No. 216. University of Auckland, Auckland (1980)

Sheppard, D.M.; Odeh, M.; Glasser, T.: Large scale clear water local pier scour experiments. J. Hydraul. Eng. **130**(10), 957–963 (2004) 20.

Mohamed, T.A.; Noor, M.M.; Ghazali, A.H.; Huat, B.K.: Validation of some bridge pier scour formulae using field and laboratory data. Am. J. Environ. Sci. **1**(2), 119–125 (2005)

11th International Civil Engineering Congress (ICEC-2020)
"Integrating innovation and sustainability in Civil Engineering"
December 22-23, 2017, Karachi, Pakistan.

A Ninth Century Earthquake induced Landslide Breaching And Downstream flood impact assesment on River Jhelum

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Abstract

In the ninth century an earthquake-induced landslide across the Jhelum River SW of Baramulla flooded the Kashmir valley to a depth of 15-25 m impounding in excess of 9-22 cubic km of flood waters before it was breached by Suya, a medieval engineer at the behest of Avantivarman, the ruler of Kashmir. The maximum stored volume was more than twice the capacity of the Mangla Dam. We calculate that the ensuing flood in the Jhelum persisted for >6 months with a discharge 50-200% greater than historically recorded levels. This initial surge locally increased levels by >40 m in the narrow Jhelum valley which after a few weeks reached steady state elevated levels of 25 m above Muzafferabad. The flood extended to the confluence with the Indus River and beyond, and would have been responsible for the transport of rock with dimensions exceeding 4 cu m to the plains. At old Mirpur, beneath the Mangla Reservoir, the flood levels would have exceeded 5 m. The history of the Jhelum River between the visit of Alexander the Great and the Muslim invasion of India is obscure, but the 883AD flood and its prehistoric predecessors would have had a devastating effect on settlements near Uri, Muzafferabad, Mirpur and Jhelum.

Keywords:
Flood Modelling, Digital Elevation Model, Landslide, Earthquake

1. Introduction

In 883AD, an earthquake in Kashmir triggered a landslide that impounded the River Jhelum and flooded the Kashmir Valley. Kalhana's Rajatarangini provides abundant details about how the ninth century engineer Suya both cleared the natural dam, drained the valley and instituted numerous irrigation works. Due to that blockage the whole Kashmir Valley inundated till Anantang (Bilham and Bali 2014). Many of the villages submerged and thousands of the people died. The total area of the Kashmir Valley is 15000 km^2. The Kashmir Valley comprises of the five rivers and numerous minor streams contribute to the Jhelum flow. The discharge rate coming towards the Kashmir Valley is strongly seasonally dependent .There have been 2000 dam failures since the 12 Century AD (VP Singh Dam Breaching Technology). HecRAS is the 1D and 2D unsteady flood routing model that can be used to route the flood (Lea, Yeonsu et al. 2019). Due to its modelling capabilities HecRAS has been used in this Research.

2. Objectives of the Study

Due to hazardous threats pose to human lives, infrastructures, floodplains, and livestock by dam failures, precision of dam break flood magnitude and propagation time at different downstream locations of the dam are essential for mitigation measures. The specific objectives of this research are:

- List of all the settlements that got effected by the Ninth Century Earthquake Induced Landslide
- Estimates of the 25 m high maximum flood.
- Estimates of what happened below the Indus confluence.
- Graph for emptying time at different flood levels based on the slowing in discharge rate with time.
- How much days required for the flood wave to reach the plains.

3. Materials and Methods

The Methodology of the study comprises of the following steps:

3.1 Phase 1: The initial phase of the study based on the extensive Literature review and compilation of the data sets.

3.2 Phase 2: The second phase comprises of the geo-referencing and digitization of the maps and acquiring of the datasets which includes Precipitation data, topography data and other physical parameters. Each Map is of different resolution and scale. Due to the low resolution of each map, digitization was quite tiresome.

3.3 Phase 3: DEM modification and reconditioning was the next step in the process. Excel, AutoCAD and Civil 3D has been utilized in order to modify the profile of River and remove sharp peaks and artefacts from it to get the accurate results.

3.4 Phase 4: The last phase is the most time consuming and significant phase of the study. It comprises of the selection of the model, simulation and analyses of the results followed by the justification of the results based on the Kashmir Valley Volume impounded due to the blockage caused by the Landslide Dam.

3. Analysis and Results

The Rainfall Depth was obtained from the different Return Periods and Hydrographs were developed on the basis of that. Flow corresponding to the Rainfall Depth was calculated by applying different Statistical approaches. These Hydrographs were simulated and Volume of the Lake filled by these Hydrographs were checked corresponding to the Discharge.

3.1 Calculated Volume and Stage of the Kashmir Valley

3.1.1 1st Simulation Result

The second Hydrograph was simulated in HEC-RAS. This Hydrograph was developed by increasing the Discharge. The Volume of the Lake was obtained as 11.63 Km3 < 22.7 Km^3, hence there is the need to increase the Discharge to completely fill the Valley up to 1600m.

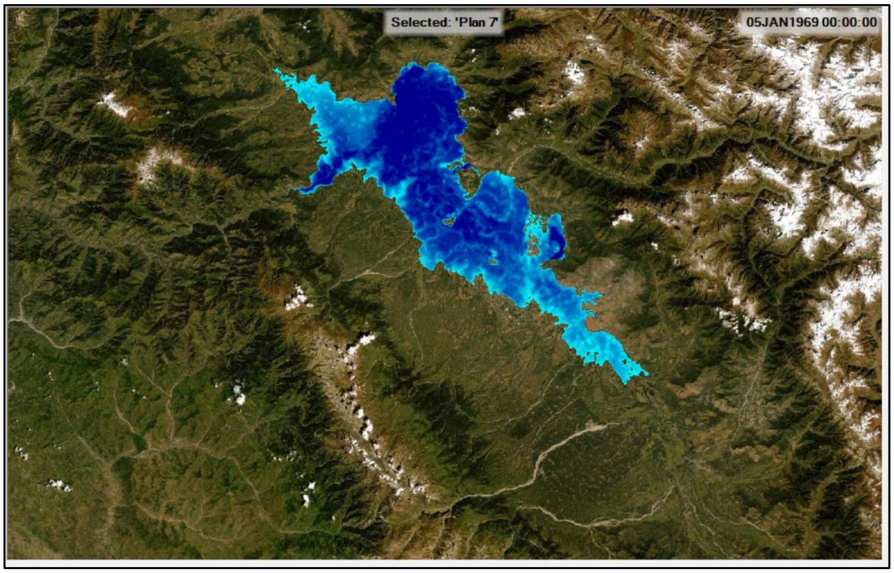

Figure-5: 2ND Simulation Result

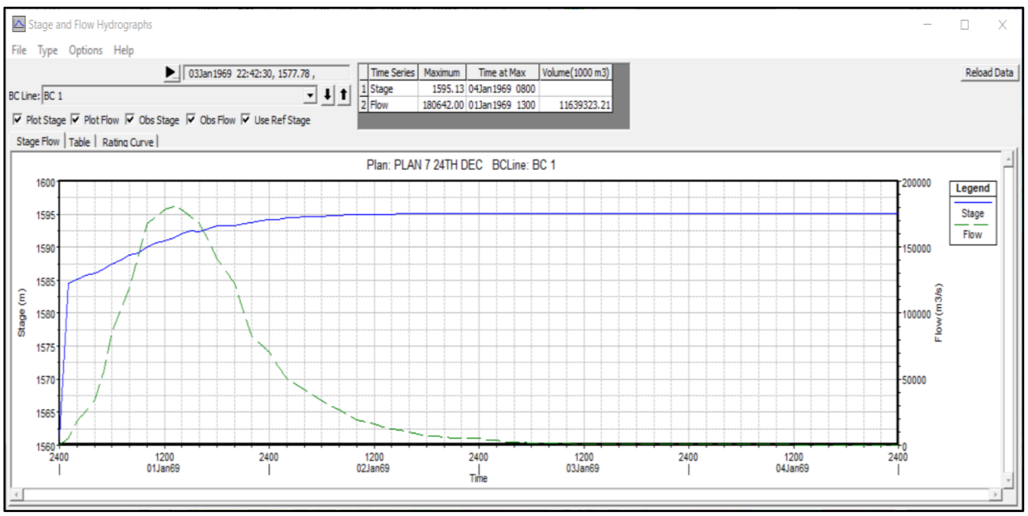

Figure-6: 2ND Simulation Hydrograph and Stage Curve

3.1.3 2nd Simulation Result

The last plan for the Hydrograph developed by increasing 20 % of 100 years return period of Rainfall was analysed and the Valley of Kashmir was completely filled by this Hydrograph up to 1600 m elevation with the Volume of 22.4 Km^3 which is approximately equal to 22.7 Km^3.The Peak Discharge of more than 200,000 Cumecs was required to filled the valley completely up to 22.7 Km^3. The area under the curve of the Hydrograph is equal to 22.4 Km3.

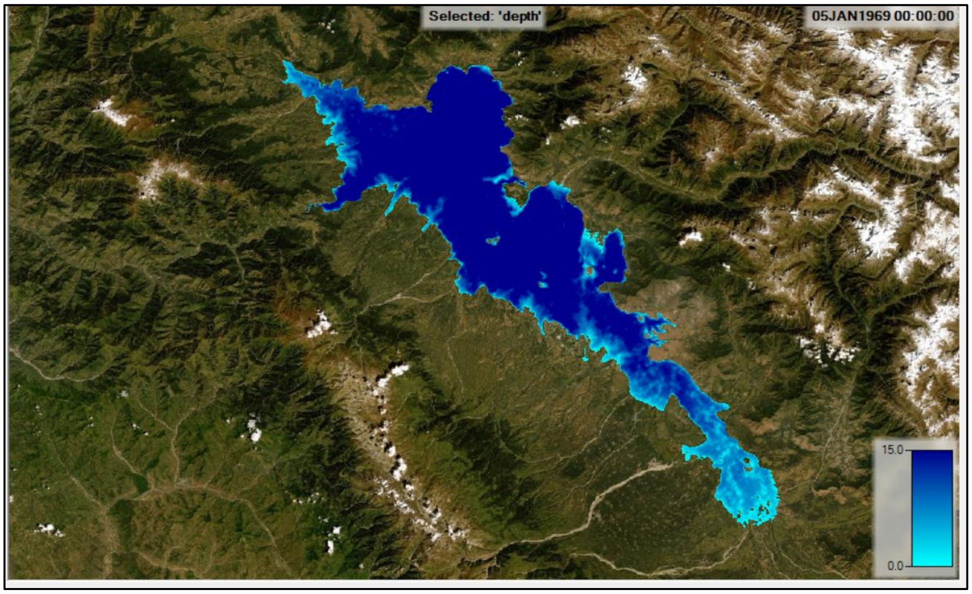

Figure-8: 3rd Simulation Result

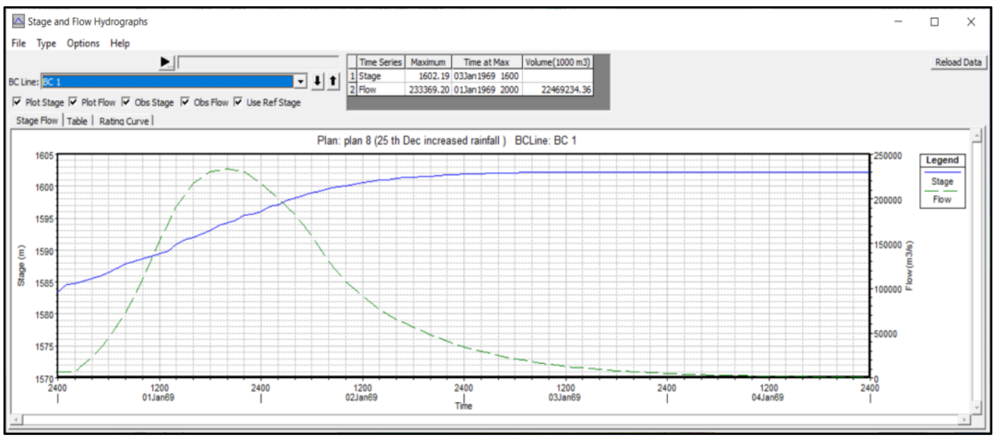

Figure-9: 3rd Simulation Hydrograph and Stage Curve

4. Inundated Villages, Depth and Velocity at Each Location

The downstream villages and areas effect by the Flood wave velocity and depth. Some of the locations identified in the downstream area, corresponding depth and Velocity has been observed at these locations. After simulation of the results, following settlements got inundated due to the flood. Using Manning Coefficient for stream bed as 0.045.

Table 1: Flood wave depth and Velocity at downstream Locations of Kashmir Valley

S.No	Location Name	Latitude and Longitude	Velocity (m/s)	Depth(m)
1	Khadinyar	34°11'44.61"**N** & 74°17'45.33"**E**	3.4	35
2	Dyargul	34°11'15.62"**N** & 74°18'20.10"**E**	2.5	27
3	Gantamula	34°10'53.20"**N** & 74°16'18.99"**E**	12.2	32
4	Minj Gram	34°10'34.41"**N** & 74°15'42.12"**E**	11	31

6	Nagnari	34°10'30.20"**N** & 74°15'0.25"**E**	12.1	29
7	Gantmulla Payeen	34° 8'34.73"**N** & 74°11'18.63"**E**	18	23.58
9	Darah Goolan	34° 7'46.78"**N** & 74° 6'33.20"**E**	16	24.35
10	Uri	34° 5'12.48"**N** & 74° 1'59.57"**E**	18	27
12	Muzaffarabad	34°21'37.31"**N** & 73°28'11.16"**E**	8	33
13	Hattian Bala	34°10'13.78"**N** & 73°44'34.70"**E**	18	25.7
15	Kalas	34°12'8.51"**N** & 73°29'52.85"**E**	11	42
16	Mangla Dam	33°12'26.31"**N** & 73°44'19.17"**E**	0.36	14

4.1 Khadinyar Location

The first location of Khadinyar was assessed and depth of the flood was more than 30 m with the velocity more than 3 m/s. The following graphs shows the water depth in m at 34°11'44.61"**N** & 74°17'45.33"**E,** starting at 0.5 days following breach of 25 m high landslide dam at Baramulla. The initial rise of 35 m can observed after 7 day, which after 5 days settle down gradually to 5m flood level which continue to decay in the next month of Feb. The maximum velocity of the flood wave was measured as 6m/s which slows down as the depth decreases.

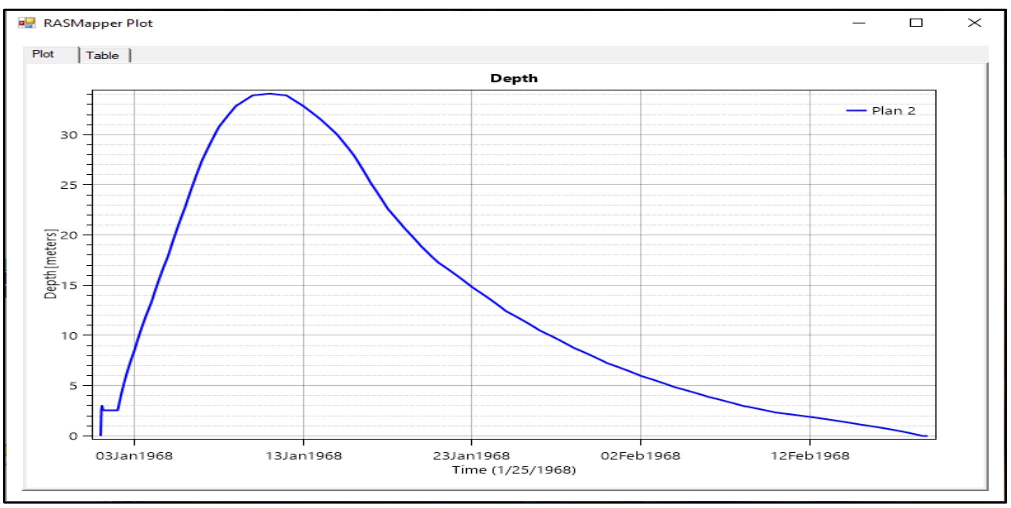

Figure-10: Khadinyar Location (Variation in the Depth of the Flood)

4.2 Muzaffarabad

The graph shows that the water level depth in m at the location of 34°21'37.31"**N** & 73°28'11.16"**E.**The initial surge was more than 30m which was observed after the 10 days followed by the breaching of the Landslide dam of 25m.

At the location of the Muzaffarabad the depth which was observed was more than 25 m and the Velocity is 2.5 m/s. The depth decreases in second moth and attain the depth of 9 m.

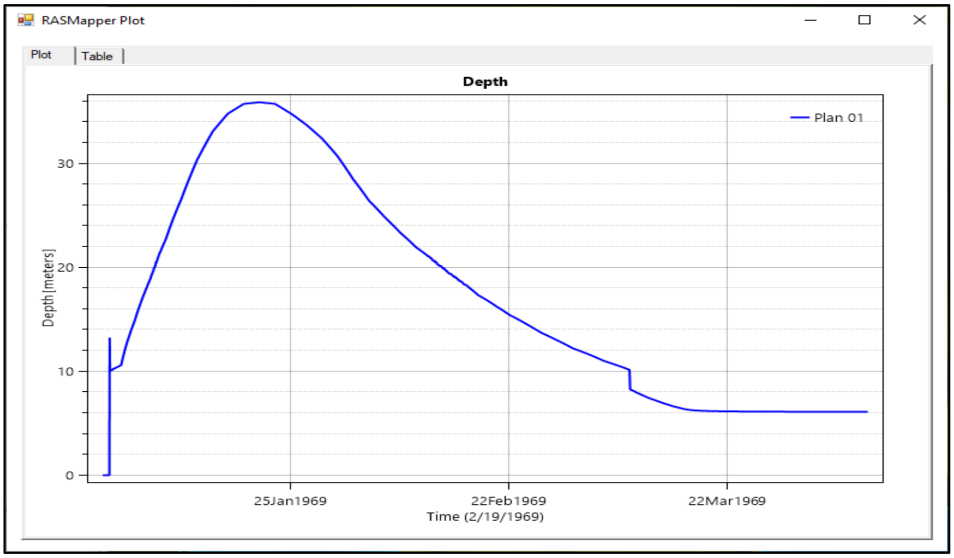

Figure-11: The depth of the Flow at Muzaffarabad

4.3 Mangla Dam

The last location was Mangla Dam. The following graph shows that the water level depth in m at the location of 33°12'26.31"**N** & 73°44'19.17"**E.**The initial surge was more than 15 m At the location of the Mangla Dam and the maximum Velocity was 0.5 m/s.

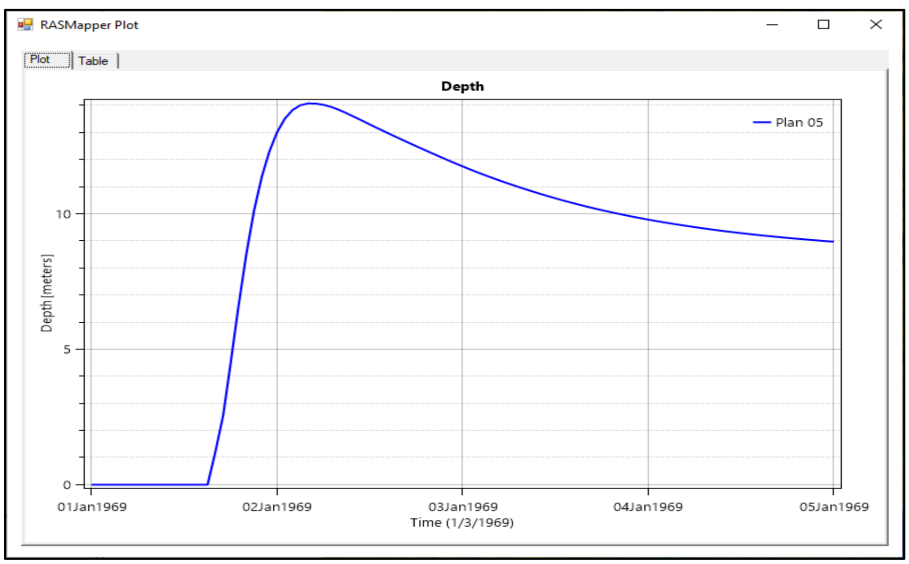

Figure-12: The Depth of the Flow at Mangla Dam

The above figure has shown the Flood wave velocity and depth at some of the Locations of the River Jhelum., where as it can be observed that the impact of the Flood is huge with very high velocity.

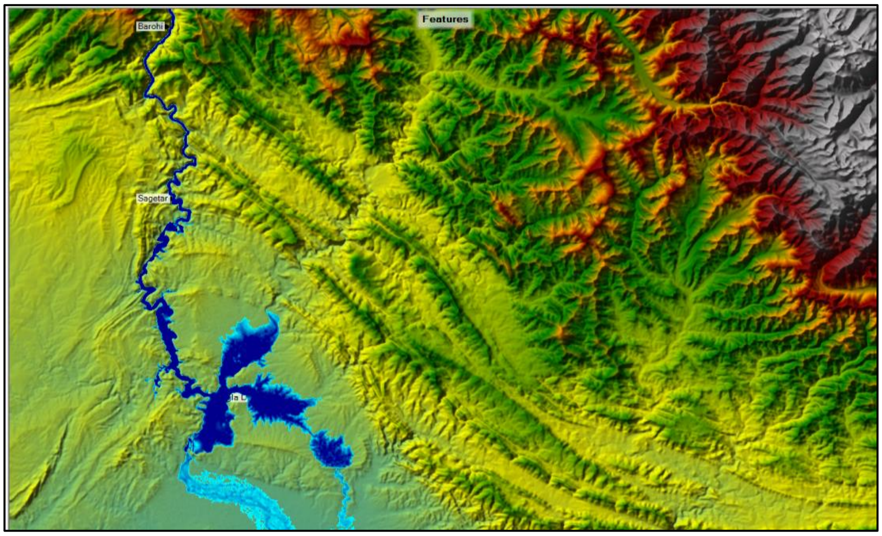

Figure-29: Inundation caused due to the Dam Breaching of Kashmir Valley

5. Justification of the Results

The flood volume of 22.7 Km3 has been used in order to assess the flood 1.3 Months will be required to cater this flood and filled the valley up to 22.7 Km3 and breach it the arrival time of the flood wave will be in 1.3 Months. The justification was based on the calculation

Table: Justification for the Peak Flood assessment breaching

Volume (Km^3)	**22.7**
Flow (m^3/s)	200,000
Time (Months)	1.32
Formulae	Q = V/T
Volume (m^3)	22700000000
Time (secs)	114048
Discharge (m^3/s)	200,000
Calculated Volume (m^3)	22809600000

6. Conclusion

The concept of the Kashmir Valley acting as a large reservoir with low hydraulic head , being suddenly permit to discharge freely down the Jhelum River southward. In geological time Kashmir Valley has flooded various times. The significant Dam Breach analysis of the Earthquake induced Landslide of the Jhelum River which was simulated with the help of HecRAS to classify the hazards towards the downstream areas. The flood assessment for the ninth century earthquake induced landslide has broaden the understanding for the impact of the Flood damages. The Flood Hydrograph was developed by different techniques and cross check by the different empirical equations. The increase of 100 Yrs of Rainfall up to 20% has caused the valley to completely filled up to 22.7 Km3. The flood wave travels up to the Mangla Dam and have a different depths and Velocity at each location. According to the Results obtained, the settlements located in downstream areas were likely to get effected the most, due to the Dam failure .The major impacts of the Dam Failure are economic loss, loss of livestock, Destruction of the Agriculture Lands and sedimentation in downstream areas.

References:

Bilham, R. and B. S. Bali (2014). *"A ninth century earthquake-induced landslide and flood in the Kashmir Valley, and earthquake damage to Kashmir's Medieval temples.*" Bulletin of Earthquake Engineering **12**(1): 79-109.

Lea, D., et al. (2019). "*Case Study of HEC-RAS 1D–2D Coupling Simulation: 2002 Baeksan Flood Event in Korea.*" Water **11**(10): 2048.

Singh, V. P. *Dam Breach Modeling Technology* (Vol. 17): Springer Netherlands.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Sustainability Assessment of a House by using LEED and BREEAM

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Abstract

The construction industry has many adverse effects on the environment. Few organizations have been educating people about these ill-effects of construction activities for awareness among people. The carbon footprint by building sector is a major concern in this regard. Thus, requiring green building concept to overcome a major effect on climate change. Green building rating systems are one of the key tools required for such sustainable endeavors. The current work reviews two of the leading tools, LEED and BREEAM by adopting a double story house. The case study has been gauged on these standard benchmarks according to the credit points systems of these two tools and has been observed that the triple bottom line of sustainability i.e. Environment, Social and Economy needs to be focused equally in the adopted rating systems.

Keywords: Carbon footprint, green buildings, green building rating systems, triple bottom line.

1. INTRODUCTION

Green Buildings concern both, the structure and the application of processes that are environmentally answerable and resource-efficient during the building life cycle i.e. from planning to design, construction, operation, maintenance, renovation and demolition. Green buildings generally have a higher upfront cost but on the other way, they provide numerous paybacks (Zhang 2014). The perceived level of sustainability is linked to the established value of the building (Myers and Reed 2008). Building rating tools can possibly play a vital role in assessing the buildings. Vierra, S. (2014) assessed that there are approximately 600 green building rating tools worldwide. Recent progress in green building rating systems has directed a number of comprehensive systems that consider many features of sustainability assessment. Due to the uniqueness of the construction industry by its nature, the concept of sustainability should be discussed in a construction industry framework. For which, four parts of environmental sustainability are measured i.e. waste management, water consumption, pollution and land use. Health, education, culture and safety are recognized as the social aspect of sustainability. thermal comfort, user control, acoustic comfort, ventilation, security, fire security, internal layout and adaptability, ease of maintenance, noise protection, and seismic safety are also involved in social sustainability. LEED (Leadership in Environmental and Energy Design) and BREEAM (Building Research Establishment's Environmental Assessment Method) are the two widely used rating systems for the performance of green buildings. These green building rating tools check the features in building: indoor environmental quality, water efficiency, energy efficiency, materials and resource, emissions and innovations and allocates a rating level. A comparison of other green building rating systems with LEED found that LEED is more rigorous in its default parameters but not addressing cost parameters (Jingwei et al. 2011). Existing green building rating systems face challenges associated with their assessment method and undefined framework which may ultimately limit their role and reliability and application in the decision-making process (Kang, 2015). Green building rating systems are basically point allocating systems which determine credits, regarding their consequence on the environmental loads' severity (Ali and Al Nsairat 2009).

However, rating systems show variation in their point system which reflects their topographical and social uniqueness. In terms of main categories green building rating tools are not homogeneous (Mattoni et al., 2018). So, there are few variations to allow for climate and cultural changes within each definite rating system. The green building rating tool is more criticized by its cost that affects its development (Zhang 2014). A number of studies have critically compared them in terms of their completeness and precision of their assessment criteria (Chen et al. 2015). Ismaeel (2016) has claimed that green building rating tools can act as operative project management and industry tools for green buildings. Shan and Hwang (2018) stated that for awareness of sustainable green building rating tools have played a significant role. A green building not only minimizes the cost and environmental impact throughout the life cycle of the building but also satisfies the basic building code requirements (Ali and Al Nsairat 2009).

The current work initially reviews the assessment for two green building rating tools to observe the comparative adoption in the construction sector. The construction industry perceives the protection of natural resources stands for environmental sustainability and social sustainability discussed the social well-being of residents (Markelj 2014). On the basis of ecological principles, green buildings are built, and the effective use of natural resources is without compromising robust facilities (Kibert 2013). The concept of sustainability needs to be implanted in all human activities to mitigate the impact of global warming and climate change (Khan, et al. 2017). Environment is polluted by construction due to the carbon emission day by day. Sustainability required in construction industry to tackle this aspect. Then to check the level of sustainability of a construction unit some green building rating tools are needed. So, here two rating systems are chosen for sustainability assessment of a house. To the best of auther knowledge no study has been conducted on sustainability of a house using LEED and BREEAM. A brief background of the two green building rating systems, provided in table 1.

Table 1: Background of LEED and BREEAM

Rating system	Country	Year	Application	Scope of Assessment
LEED (Leadership in Environmental and Energy Design)	United States (US)	1993	New construction, Existing building, Homes, operation and maintenance	Sustainable sites, Water efficiency, Material and resources, Energy and Atmosphere, Location and Transport, Indoor environmental quality, Regional priority, innovation
BREEAM (Building Research Establishment Environmental Assessment Method)	United Kingdom (UK)	1990	New construction, Existing building, Homes, communities, infrastructure.	Management, Health, and wellbeing, Material and resources, Energy and Atmosphere, Water efficiency, Waste management, Pollution, Innovation

2. METHODOLOGY:

There are many green building rating tools for the assessment. Two green building rating tools are used for sustainability assessment of a house i.e. LEED (Leadership in Environmental and Energy Design) and BREEAM (Building Research Establishment's Environmental Assessment Method). These rating systems check the performance of the house according to their standards. Fig 1. presents methodology of the research work is in a graphical format

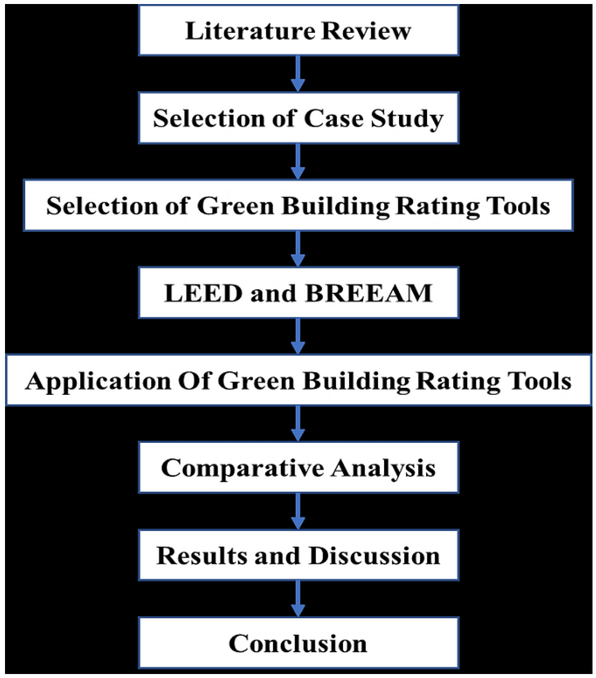

Figure 1: Methodology of Research Work

2.1 Case Study:

A two-story house has been selected located in the capital city as a case study. The area of our concerned house is 5445 ft^2. The house is a frame structure with a single basement, ground floor and first floor. The outer face of the house is finished with modern architectural features with plain glass on the front elevation.

Figure: 2 Elevation of selected house

2.2 Assessment Criteria:

For assessment criteria, two green building rating systems are used i.e. LEED and BREEAM. For LEED we used (LEED v4.1) and for BREEAM we use (BREEM 2014, 2.2).

2.2.1 LEED:

LEED checks the sustainability of buildings by awarding different points to building according to its standard. The points show the sustainability level of the building. LEED points are fragmented down into night sub-categories: Sustainable Site, Water Efficiency, Energy & Atmosphere, Materials & Resources, Indoor Environmental Quality, Locations & Linkages, Awareness & Education, Innovation in Design and regional priority. Table (2) shows the LEED certification criteria (LEED v4.1).

Table 2: Point distribution criteria for (LEED v4.1)

Category	Points
Certified	27-33
Silver	34-39
Gold	40-52
Platinum	52-70

2.1.2 BREEAM:

Table 3: Point distribution criteria for (BREEAM 2014, 2.2)

Category	% of threshold
Unclear	Below 25% of threshold
Pass	25%-40% of threshold
Good	40%-60% of threshold
Very Good	60%-75% of threshold
Excellent	Above 75% of threshold
Outstanding	Above 85% of threshold

BREEAM ratings are determined by achieving a set percentage of threshold points. Buildings must attain at least 35% of the threshold if qualify. 39% of the clients stated that BREEAM drove them to invest in innovation. Table (3) shows BREEAM certification criteria (BREEAM 2014, 2.2)

3. RESULTS:

The results are obtained after applying LEED and BREEAM rating tools to a two-story house. Both rating tools have different points of awarded criteria. Sustainability aspects (environmental, social and economic) can also be shown by the rating tools, how much they give priority to each aspect.

3.1 LEED:

The 8 major categories of (LEED v4.1) are shown in table 4. The standard available points of each category and points obtained after applying the LEED rating system on a two-story house are also shown. According to (LEED v4.1) 51 points are awarded to our concerned house and maximum points are given to energy efficiency and minimum points are given to regional priority and innovation.

Table 4: Points awarded by (LEED v4.1)

Category	Points available	Points awarded
Sustainable sites	10	4
Water efficiency	12	6
Material and resources	13	7
Energy and Atmosphere	34	16
Location and Transport	15	9

Indoor environmental quality	16	7
Regional priority	4	1
innovation	6	1
Total	110	51

3.2 BREEAM:

The 8 major categories of (BREEAM 2014, 2.2) are shown in table 5. The standard available points of each category and points obtained after applying the BREEAM rating system on a two-story house are also shown. According to (BREEAM 2014, 2.2) 57 points are awarded to our concerned house and maximum points are given to energy efficiency and minimum points are given to waste management.

Table 5: Points awarded by (BREEAM 2014, 2.2)

Category	Points available	Points awarded
Management	12	8
Health and wellbeing	17	10
Material and resources	8	4
Energy and Atmosphere	43	24
Water efficiency	11	6
Waste management	3	1
Pollution	6	2
Innovation	10	2
Total	110	57

3.3 Comparative Analysis:

By applying LEED to our concerned house, our house got 51 points according to (LEED v4.1) and give first priority to energy efficiency. According to LEED, our house is 'GOLD' certified because the total points obtained are 51. BREEAM also gives first priority to energy efficiency and award 57 points to housing and classified as the 'GOOD' category. According to LEED, our concerned house is less innovative but for BREEAM our house is marginally more innovative. It can be seen that waste management is not considered in LEED but for BREEAM waste management is considered. Indoor environmental quality is one of the concerned categories in LEED, while BREEAM has not this category. Pollution is considered by BREEAM. So, out of eight categories, four categories are common in both rating systems.

3.4 Comparison Of Triple Bottom Line Of Sustainability:

A comparison of the triple bottom line of sustainability can be observed by the points awarding distribution of LEED and BREEAM. Both rating systems give maximum priority to the Environmental aspect. The economic aspect of sustainability is not as important as the environmental aspect according to LEED and BREEAM. Both rating systems have no category of focus that relates to the economic aspect. The environmental aspect of sustainability has some value in these two rating systems as some categories relate to the social aspect.

4. DISCUSSION:

These two rating systems differ in many ways. Each system covers a set of criteria that may or may not differ from the others. Systems that take more aspects in their consideration are more likely to broadly cover the different aspects of sustainability than those with fewer criteria or items. By comparing two rating systems from the aspect of the triple bottom line of sustainability, the study observed results that both concerned rating systems majorly focus on the environmental aspect.

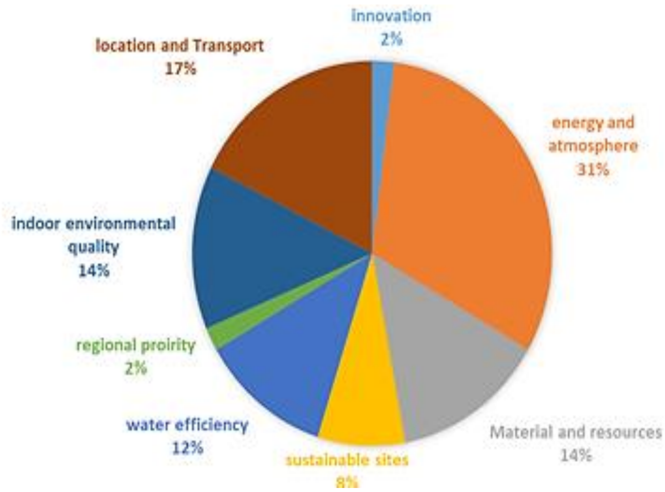

Figure: 3 Percentages of points awarded by (LEED v4.1)

After considering all of the efforts to increase sustainability in the built environment, it is critical that the goal of more green buildings is not hindered by the absence of the truly sustainable rating system. In short, by adopting these green building rating systems, we will know that the building is sustainable or not according to the standard. Figure 3 shows that in LEED the maximum awarded percentage is 31% which is energy and atmosphere. Figure 4 shows that in BREEAM also maximum awarded percentage to energy and atmosphere which is 42%.

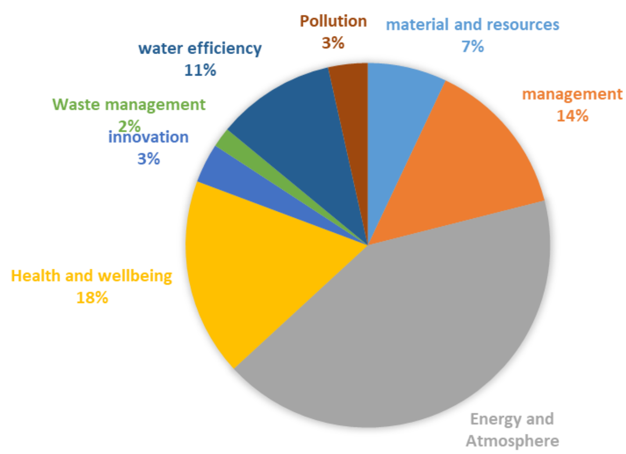

Figure: 4 Percentages of points awarded by (BREEAM 2014, 2.2)

5. CONCLUSIONS:

Green building rating systems have the common characteristics of comprising several entries or very concise categories on the design and planning of the building, which decrease local and global environmental impacts caused by them. Each rating system gives certification to the two-story house according to its own standard.

Following conclusions can be drawn from the conducted study:

- According to LEED, our concerned house is GOLD classified with 51 points and BREEAM classified it as GOOD with 57 points.
- Energy is the major concern in both green building rating systems. In LEED the energy and atmosphere category got 31% and in BREEAM it got 42 %.
- Innovation has minimum credits in both cases i.e. 1% and 3% in LEED and BREEAM respectively.
- By comparing the results of both systems, the case study has more sustainable features our house

is more sustainable according to BREEAM.

These green building rating systems have successfully applied to a two-story house. Each system has some special focusing points related to green building certification. Further research should be done by using other green building rating tools other than these two and check how they focus on three pillars of sustainability.

Acknowledgments:

The authors would like to thank every person/department who helped throughout the research work, particularly Irbaz Hasan and the CE department of CUST. "The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged."

References:

Bourdeau, L. and Huovila, P. (2008). "sustainable development and future of construction." *Journal on building research and information,* 21(09), 331-339.

Bih, K. and Nguyen, H. (2011)." Comparative review on five sustainable rating *systems", procidia engineering,* 21, 376-386.

Candace, S. and Architect, G. (2008)." Sustainable rating system around the world", *council on a tall building and urban habitat,* 25, 448-459.

Chandratilake, W. and Dias, Q. (2013)." Sustainability rating systems for buildings: comparisons and correlations *"Energy,* 59, 22-28.

Chen, X., Yang, H. and Lu, L. (2015). "A comprehensive review of passive design approaches in green building rating tools." *Renewable and Sustainable Energy Reviews,* 50, 1425-1436.

Datien, D., Ghaffarianhoseini, A. and Nicola, N. (2017)." A critical comparison of green building rating systems*", Building and environment* 123, 243-260.

Fawcet, W. and Hughes, M. (2012). "A flexible strategy for long term sustainability." *building research and information,* 545-557.

Illankoon, I., Tam, V. and Le, K. (2016). "Environmental, Economic and Social parameters in International Green Building Rating Tools."

Mattoni, B., Guattari, C., Evangelisti, L., Bisegna, S., Gori, P. and Asdrubali, F. (2018). "Critical review and methodological approach to evaluate the differences between international green building rating tools." *Renewable and Sustainable Energy Reviews,* 82, 950-960.

Richard, R., Anita, B. and Sara, W. (2009)." International comparison of sustainable rating tool*." Sustainable real estate,* 01, 1-22.

Rebecca, C. and Retzlaff, Z. (2008). "Green Building Assessment Systems, A framework and comparison for planners." *Journal of American planner's association,* 505-519.

Shan, M. and Hwang, B. (2018). "Green Building Rating Systems: Global Reviews of Practices and Research Efforts."

Silverio, Marcos, Ignacio, Jose A. (2017). "Green buildings rating systems and their application in the Mexican context*", Theoretical and empirical researches in urban management* 12, 20-32.

Sev, A. (2009). "A comparative analysis of building environmental and suggestions for regional adaptations. *"Civil engineering and environmental systems.* 28(03), 231-235.

Varma, S. and Palaniappan, S. (2019)." Comparison of green building rating schemes used in North America, Europe and Asia.", *Habitat International,* 89.

Waidyasekara, K. and Silva, M. (2013). "Comparative study of green building rating systems: in terms of water efficiency." *Economic sustainability in construction,* 14(15), 108-117.

Walaa, S. and Ismaeel, R. (2018).” Drawing the operating mechanism of green building rating systems”, *Cleaner production,* 213, 599-609.

Xiangzang, C. and Zhan, X. (2019). “Asian green building rating tools: a competitive study on scoring method on quantitative evaluation systems”, *Cleaner production,* 218, 880-895.

Yashwanth P. and Kona, M. (2019),” A comparative study on green building rating systems in India in terms of energy and water*”, Science and Technology,* 16, 117-129.

Zhan, C. and Cheng, j. (2019).” A review of renewable energy assessment methods in green building and green neighborhood rating system*”, Energy and Buildings,* 195, 68-81.

11th International Civil Engineering Conference (ICEC-2020)
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March13-14, 2020, Karachi, Pakistan.

Resource Procurment and handling Issues Generating Waste in Construction Projects

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Abstract

Construction project around the globe are facing a serious issue of waste generation. Construction waste has not only affected the cost of the project but also is a threat to environmnet. Hence, it is very cruicail to control waste generation on construction projects. These wastages are generated by several sources. In order to identify these sources of waste generaion this paper focused on qualitative study. Its scope if limited to study resource related issues causing waste generation. A comprehensive literatre resulted in identifying 20 attributes of waste generation which were investigated through interview of 6 experienced practitioners. Statistical analysis of the data gathered through interview highlighted that out of 20 attributes, 19 attributes have RI value above 0.5 and have good level of relevancy of with construction industry of Sindh province while only 1 factors i.e. theft and vandalism of the resources are reported as less relevant.

Keywords

Construction Waste, Waste Factors, Resource Related Issues, Resource Management, Resource Procurement.

1. Introduction

Construction waste can be defined as any loss resulting from construction activities which may cause direct and indirect costs and consume time but does not add any value to the final product. Increased construction development has increased waste generation in developing countries like Pakistan. Construction wastes are classified as physical and non-physical where waste of materials is regarded as physical waste whereas waste of time of cost is non-physical waste (Akhund et. al. 2019). Construction waste is a major problem at almost every construction site. Construction and demolition activities produces a large amount of construction waste. It has an adverse impact on time, cost, productivity and sustainability (Akhund et. al. 2018). Construction waste has a serious impact on the time, cost, productivity and economy of the industry as well as it has an effect on the environment. It was concluded that execution phase of project life cycle generate more waste as compare to other phases of the project life cycle. The significant waste generation factors are poor labor skills and ordering errors (Akhir et. al. 2013).

Waste generation results from poor management system at construction site, unsuitable methods of materials storage, unskilled labors, poor site layout, poor communication among stakeholders and improper planning (Akhund et. al. 2017). The waste generation has been increasing and thus becomes a burden to landfills which is a usual way of disposals. Awareness need to be created among the stakeholders about the importance of waste generation. Main challenge is to the managing organizations for not giving attention to the proper waste management (Essa et. al. 2015).

As a developing country Pakistan is also facing a serious problem of construction waste generation. Today waste generation is a common phenomenon in the construction industry. It has adverse impacts on the surrounding and environment. Moreover, it has also negative effects on costs productivity, time, social and economic aspects of the industry (Akhund et. al. 2018). Traditional construction methods in Pakistan particularly in the province of Sindh are the reason of massive generation of construction waste. In Sindh many construction projects are going on which are producing huge amount of construction waste which then disposed off illegally to the country side lands (Memon et. al. 2016). Previously research has been done to generally find out the factors contributing to the waste generation in the construction projects. The causative factors of the waste generation are last minute design changes, unskilled labors, shortage of equipments, poor management skills, poor materials management, wrong material delivery procedures, unpredicted incidents at site and lack of knowledge on construction projects (Nagapan et. al. 2012). Hence, this study is carried out to identify the attributes causing waste generation. However, the scope of this study is limited to resource related issues and construction works of Sindh.

2. Review of Literature

Construction industry uses massive quantity of natural resources. At the same time, it is a key producer of waste also (Manowong, 2012). Waste is any unnecessary material (Mhaske et al. 2017) which may include building debris, rubble, earth, concrete, steel, timber, and mixed site clearance materials (Shenet al. 2004). United States Environmental Protection Agency (USEPA), (2009) highlighted that in 2003 approximately 170 million tons of waste was generated from building works. Waste generated is major cause of revenue loss also and 21–30% of budget overrun occurs due to materials wastage (Ameh and Daniel, 2013). According to Bossink & Brouwers (1996)9% (by weight) of the materials go as waste. In addition, it has been found that those contractors who produced higher waste faced a 10% disadvantage in tendering (Guthrieet al. 1995).The construction waste poses serious environmental issues. This problem is universal and equally common in developed and developing regions. As an example, cases of few countries are discussed below..

- Balkans: construction and demolition waste, which comprises of glass, concrete, bricks, asphalt, plastic, wood, metals, and other materials, has a representation of 25 - 30 % in all generated waste (Jelić et al. (2018).
- Malaysia: Due to C&D waste generated has caused increase in illegal dumping (Begum & Pereira, 2011). Major types of waste were timber, bricks, packaging waste and concrete (Foo et al. 2013). Wahi et al. (2016); stated that in the year 2012, 25600 ton per day construction waste was generated in execution phase due to the construction activity. Similarly, in 2014, every day 26000 tons of CW was generated (Esa et al. 2017).
- United States: 39% increase in C&D waste has been observed since 2007 (Tam et al. 2016; & Won et al. 2016).
- Canada: Thirty five percent of the space in Canada's landfills is taken up with construction wastes and debris (Tam et al. 2016).
- China: Wang et al. (2008); reported that concrete, cement, brick, timber, tile, steel, and aluminum wastes are common waste type in construction. Average Waste Generation Rate (WGR) is ranged from 3.275 to 8.791 kg/m2 (Lu et al. 2011). Studies revealed that in 2008 and 2009, and in 2013

one billion tons of waste was produced, out of which 74% was construction demolition waste (Wahi et al. 2016).

- India: Due to construction activities, 10-12 million tons is produced every year (Ponnada and Kameswari, 2015).

In building works there are several activities and attributes which cause waste generation such as excavation/ demolition, materials waste, packing (Johnston & Mincks 1995); temporary works (Jaillon et al. 2009); and other external causes, for example, theft and vandalism (Bossink & Brouwers 1996); scaffolding, concrete work, materials handling and hoardings (Jaillon et al. 2009; Wang et al. 2008). However, the quantity of waste generation varies significantly from project to project (Jaillon et al. 2009). Furthermore, the amount of waste generation depends on the following factors; the type of building structure, the technology used to construct it, the schedule and performance of the construction process (Hao et al. 2008); the period of construction, the method and techniques used for construction, the main materials used (Kourmpanis et al. 2008). Jaillon et al. (2009) mentioned that construction method play important role in waste generation while Katz and Baum (2011) mentioned that there is no correlation between construction methods and the amount of waste generation.

Even though no comprehensive list of factors is identified as cause of waste generation, researchers have highlighted different sources of waste generation as: design, procurement, materials handling, operation, residual and other (Bossink & Brouwers 1996; Gavilan & Bernold 1994). Together with technical attributes, the attitudes and behaviors of the practitioners of construction is also key source of waste generation (Kulatunga et al. 2006). Chen et al. (2002) also highlighted that workers' attitudes towards construction operations have a direct impact on construction waste generation with incorrect or careless use of materials by workers found to be a main cause of waste generation. Thus, it can be argued that human factors related to waste management are often linked to technical factors instead of considering the former as a key consideration of waste management.

Chen et al. (2002); stressed that even though waste management is seen as a site responsibility, most waste is generated as the result of damage in transit. 19 external factors including packaging and design, besides site management were identified. Osmani et al. (2008); also acknowledged that detailing errors and design changes requested by clients cause waste generation. Katz and Baum (2011); identified that the waste accumulation pattern in construction projects is similar in the beginning and increases in quantity at the end of the project. Research based in Israel found that two-thirds of waste was generated in the last third of the construction process Katz & Baum (2011). Garas et al. (2001); studied construction projects of Egypt and reported that material waste was mainly because of late information, uncompleted design, inadequate information, poor control, unnecessary people's moves. Alwi et al. (2002b); indicated that key variables causing waste generation include design changes, slowness in making decisions, lack of trades' skill. Adewuyi and Odesola (2015); revealed that key causes of construction waste are design and documentation related issues followed by material procurement. Seze & Seze, (2017); drew attention that on projects there are less efforts observed related to waste management techniques on site which is the main reason of waste generation. A comprehensive literature review was carried out to identify resource procurment and management factors causing waste generation as summarized in table 1.

Table 1: Waste Generating Factors

S. No	Factor	Reference
1	Damage during transportation	Akhund et.al 2017, Memon et.al 2016
2	Delay during delivery	Memon et.al 2016, Nazech et al (2008), Polat & Ballard (2004), Zhao & Chua (2003), Alwi et al., (2002b)
3	Equipment failure	Arshad et.al 2018, Memon et.al 2016, Nagapan et al (2012), Osmani et al (2008), Wang et al (2008), Zhao & Chua (2003)
4	Inappropriate resource	Akhund et.al 2019, Arshad et.al 2018, Akhund et.al 2017, Nazech et

	allocation	al (2008)
5	Inappropriate use of materials	Nazech et al (2008), Alwi et al., (2002b)
6	Insufficient methods of loading	Osmani et al (2008)
7	Items not in compliance with specification	Osmani et al (2008), Garas et al (2001)
8	Left over material on site	Kofoworolo & Gheewal (2009), Wang et al (2008)
9	Material supplied in loose form	Daoud et al. (2018), Osmani et al (2008)
10	Ordering errors	Memon et.al 2016, Nagapan et al (2012), Lu et al (2011), Wang et al (2008), Polat & Ballard (2004), Garas et al (2001)
11	Outdated equipments	Alwi et al., (2002a) , Garas et al (2001)
12	Poor controlling	Nagapan et al (2012), Kofoworolo & Gheewal (2009), Polat & Ballard (2004), Zhao & Chua (2003), Garas et al (2001)
13	Poor material handling	Akhund et.al 2019, Daoud et al. (2018), Akhund et.al 2017, Memon et.al 2016, Lu et al (2011), Kofoworolo & Gheewal (2009), Nazech et al (2008), Polat & Ballard (2004), Alwi et al., (2002b) , Garas et al (2001)
14	Poor quality of materials	Memon et.al 2016, Lu et al (2011),Nazech et al (2008), Wang et al (2008), Alwi et al., (2002a), Alwi et al., (2002b)
15	Scarcity of equipment	Daoud et al. (2018), Memon et.al 2016, Nazech et al (2008), Polat & Ballard (2004), Zhao & Chua (2003), Alwi et al., (2002b)
16	Theft and vandalism	Daoud et al. (2018), Garas et al (2001)
17	Waiting periods	Zhao & Chua (2003), Alwi et al., (2002a) , Garas et al (2001)
18	Wastage due to cutting	Arshad et.al 2018, Wang et al (2008)
19	Waste resulting from packaging	Kofoworolo & Gheewal (2009), Wang et al (2008)
20	Wrong material storage	Akhund et.al 2019, Arshad et.al 2018, Daoud et al. (2018), Memon et.al 2016, Hassan et.al 2012,Nagapan et al (2012), Lu et al (2011), Nazech et al (2008), Wang et al (2008), Zhao & Chua (2003), Alwi et al., (2002b) , Garas et al (2001)

3. Methodology

This study involved qualitative method using structured interview in order to determine the relevancy of the attributes of waste generation related to construction sectors of Sindh province. Selected respondents from contractor and consultants were approached through telephone and interviewed in person. The interviewees were given list of the attribute identified through review of literature piblished gloabally and were asked to intimate the relevancy of each attribute with construction sector of Sindh using 2 point scale as 1=Not relevant, 2=medium relevant and 3=highly relevant. Thr response of the people was analyzed with the formula:

$$RI = \sum \frac{W}{A \times N}$$

Where W is the weight assignmed to each scale, A is the highest scale i.e. 3 in this case and N is total number of reponses i.e. 6.

4. Findings of the study

Interviews were conducted amongst senior practitioners of construction industry. The demographic information of the interviewee panel are shown in table 2.

Table 2: Demographic information of the respondents Involved in Interview

Position	Type of Organization	Education	Experience (Years)
Project Engineer	Contractor	BE (Civil Engineering)	17
Project Engineer	Contractor	BE (Civil Engineering)	14
Construction Manager	Contractor	ME (Civil Engineering)	22
Project Director	Consultant	Master (Project Management)	16
Project Director	Consultant	Master (Project Management)	23
Project Manager	Consultant	Master (Civil Engineering)	17

Table 2 shows that the total experience of the 06 respondents interviewed is 109 years with an average of 18.2 years. The respondents are senior employees of their respective companies. All the respondents selected for this interview section are engaged in handling large construction projects and holding managing & executive posts. The respondents have obtained civil engineering degrees while some have earned master degree in civil engineering and project management. This shows that the interviewees are capable and reliable to explore the underpinning issues related to construction waste. The results obtained from statistical analysis regarding the attributes are presented in table 3.

S. No	Attribute	1	2	3	total	RI
1	Damage during transportation	1	4	1	6	0.67
2	Delay during delivery	1	3	2	6	0.72
3	Equipment failure	0	2	4	6	0.89
4	Inappropriate resource allocation	0	4	2	6	0.78
5	Inappropriate use of materials	2	1	3	6	0.72
6	Insufficient methods of loading	2	3	1	6	0.61
7	Items not in compliance with specification	1	1	4	6	0.83
8	Left over material on site	3	1	2	6	0.61
9	Material supplied in loose form	2	1	3	6	0.72
10	Ordering errors	2	2	2	6	0.67
11	Outdated equipments	2	1	3	6	0.72
12	Poor controlling	0	2	2	4	0.83
13	Poor material handling	2	2	2	6	0.67
14	Poor quality of materials	0	3	3	6	0.83
15	Scarcity of equipment	1	2	3	6	0.78
16	Theft and vandalism	4	0	1	5	0.47
17	Waiting periods	1	3	2	6	0.72
18	Wastage due to cutting	0	2	4	6	0.89
19	Waste resulting from packaging	0	1	5	6	0.94
20	Wrong material storage	0	2	4	6	0.89

From table 3, it can be out of 20 attributes, 19 attributes have RI value above 0.5 and have good level of relevancy of with construction industry of Sindh province while only 1 factors i.e. theft and vandalism of the resources are reported as less relevant. This shows that either the resources on site are safer or the contractors purchase the resource when they are required to use.

5. Conclusion

This paper focused on identifying common resource procurement and handling related attributes causing construction waste generation in Sindh Province. It was done through detailed review of previously published literature around the world regarding subject matter. This helped in determining 20 common attributes which were enquired for relevancy with construction works of Sindh. Interviews from six experienced practitioners were conducted and analyzed statistically with relative index formula. It was found that the practitioners indicated that 19 factors have high relevancy with construction works of Sindh while only one factor i.e theft and vandalism was reported as the attribute with low relevancy.

6. References

Adewuyi, T.O., and Odesola, I.A. (2015). "Factors Affecting Material Waste on Construction Sites in Nigeria". *Journal of Engineering and Technology (JET)*, Vol. 6, No. 1, pp. 82-99

Alwi, S., Hampson, K., and Mohamed, S. (2002a). "Non Value-Adding Activities in Australian Construction Projects". Proceedings International Conference on Advancement in Design, Construction, Construction Management and Maintenance of Building Structure, pp. 270–278.

Alwi, S., Hampson, K., and Mohamed, S. (2002b). "Waste in the Indonesian Construction Project". *Proceedings of the 1st International Conferences of CIB W107 – Creating a Sustainable Construction Industry in Developing Countries*, pp. 305-315.

Akhir, N.S.M., Rahman, I.A., Memon, A.H., and Nagapan, S. (2013). "Factors of Waste Generation Throughout Construction Life Cycle", *Proceedings of Malaysian Technical Universities Conference on Engineering & Technology*.

Akhund, M.A., Memon, A.H., Memon, N.A., Ali, T.H., and Khoso, A.R. (2019). "Exploring Types of Waste Generated: A study of Construction Industry of Pakistan", *In MATEC Web of Conferences*, Vol. 266, pp. 05011, EDP Sciences..

Akhund, M.A., Memon, N.A., Ali, T.H., Memon, A.H., and Imad, H.U. (2018). "A Comprehensive Review on Waste Generating Attributes: Way Forward for Pakistan's Construction Industry". *Engineering Science and Technology International Research Journal*, Vol. 02, No. 03, pp.

Akhund, M.A., Khoso, A.R., Pathan, A.A., Siddiqui, F., amd Abbasi, I.A., (2017). "Minimization of Waste Generation at Construction Sites", *International Journal of Engineering & Technology*, Vol. 09, No. 03.

Ameh, O.J., and Daniel, E.I. (2013). "Professionals' views of material wastage on construction sites and cost overruns". *Organization, Technology & Management in Construction: An International Journal*, Vol. 5, No. 1, pp. 747-757.

Arshad, H., Qasim, M., Thaheem, M. J., and Gabriel, H. F. (2017). "Quantification of Material Wastage in Construction Industry of Pakistan: An Analytical Relationship between Building Types and Waste Generation". *Journal of Construction in Developing Countries*, Vol. 22, No. 2, pp. 19-34.

Begum, R.A., and Pereira, J.J. (2011). "C & D waste profile of the Malaysian construction industry: Need a centralized database". *In Sustainable Technologies (WCST) 2011 World Congress*, pp. 73-76

Bossink, B.A.G., and Brouwers, H.J.H. (1996). "Construction waste: quantification and source evaluation". Journal of Construction Engineering and Management, Vol. 122, No. 1, pp. 55-60.

Chen, Z., Li, H. and Wong, C.T.C. (2002). "An application of bar-code system for reducing construction wastes", *Automation in Construction*, Vol. 11, No. 5, pp. 521-533.

Daoud, A.O., Othman, A.A., Robinson, H., and Bayyati, A. (2018). "Exploring the Relationship Between Materials Procurement and Waste Minimization in The Construction Industry: The Case of Egypt". *4th NZAAR International Event Series on Natural and Built Environment, Cities, Sustainability and Advanced Engineering*, pp. 76-85.

Esa, M.R., Halog, A., and Rigamonti, L. (2017). "Developing strategies for managing construction and demolition wastes in Malaysia based on the concept of circular economy". *Journal of Material Cycles and Waste Management*, Vol. 19, No. 3, pp. 1144-1154.

Esa, M.R., Halog, A., and Ismail, F.Z., (2015). "Waste Management in Construction Industry: A review on the Issues and Challenges", *International Conference on Environmental Research and Technology*.

Foo, L.C., Rahman, I. A., Asmi, A., Nagapan, S., & Khalid, K. I. (2013). "Classification and quantification of construction waste at housing project site". *International Journal of Zero Waste Generation*, Vol. 1, No. 1, pp. 1-4.

Garas, G.L., Anis, A.R. and El Gammal, A., (2001). "Materials waste in the Egyptian construction industry". *Proceedings IGLC-9*.

Gavilan, R.M., and Bernold, L.E. (1994). "Source evaluation of solid waste in building construction". *Journal of Construction Engineering and Management*, Vol. 120, No. 3, pp. 536-552.

Hassan, S.H., Ahzahar, N., Fauzi, M.A., and Eman, J. (2012). "Waste Management Issues in the Northern Region of Malaysia". *Procedia - Social and Behavioral Sciences*, Vol. 42, pp. 175–181.

Jaillon, L., Poon, C., and Chiang, Y. (2009). "Quantifying the waste reduction potential of using prefabrication in building construction in Hong Kong". *Waste Management*, Vol. 29, No. 1, pp. 309-320.

Jelić, I., Šljivić-Ivanović, M., Dimović, S., Antonijević, D., Jović, M., Mirković, M., and Smičiklas, I. (2018). "The applicability of construction and demolition waste components for radionuclide sorption". *Journal of Cleaner Production*, Vol. 171, pp. 322-332.

Johnston, H., and Mincks, W.R. (1995). "Cost-effective waste minimization for construction managers". *Cost Engineering*, Vol. 37, No. 1, pp. 31.

Katz, A., and Baum, H. (2011). "A novel methodology to estimate the evolution of construction waste in construction sites". *Waste management*, Vol. 31, No. 2, 353-358.

Kofoworola, O.F., and Gheewala, S.H. (2009). "Estimation of construction waste generation and management in Thailand". *Waste management*, Vol. 29, No. 2, pp. 731-738.

Kourmpanis, B., Papadopoulos, A., Moustakas, K., Stylianou, M., Haralambous, K., and Loizidou, M. (2008). "Preliminary study for the management of construction and demolition waste", Waste Management and Research, Vol. 26, No. 3, pp. 267-275.

Kulatunga, U, Amaratunga, D, Haigh, R. and Rameezdeen, R. (2006). "Attitudes and perceptions of construction workforce on construction waste in Sri Lanka". *Management of Environmental Quality: An International Journal*, Vol. 17, No. 1, pp. 57-72.

Lu, W., Yuan, H., Li, J., Hao, J. J., Mi, X., and Ding, Z. (2011). "An empirical investigation of construction and demolition waste generation rates in Shenzhen city, South China". *Waste management*, Vol. 31, No. 4, pp.680-687.

Memon, A.H., Soomro, M.A., Bhangwar, S.N., Memon, A.H., amd Memon, M.U. (2016). "Consultants' Perspective on Factors Causing Construction Waste Generation in Less Developed Regions". *International Journal of Engineering Inventions*, Vol. 05, No. 06.

Mhaske, M., Darade, M. and Khare, P. (2017). "Construction waste minimization", *International Research Journal of Engineering and Technology (IRJET)*, Vol. 4, No. 7, pp. 934-937.

Nagapan, S., Rahman, I.A., Asmi, A. (2012), "Factors Contributing to Physical and Non-physical Waste Generation in Construction Industry", *International Journal of Advances in Applied Sciences*, Vol. 01, No. 01.

Nazech, E.M., Zaldi, D., and Trigunarsyah, B. (2008). "Identification of Construction Waste in Road & Highway Construction Projects". *Proceedings Eleventh East Asia-Pacific Conference on Structural Engineering and Construction (EASEC-11)*, Vol. 2008, pp. C07-02-1

Osmani, M, Glass, J and Price, A.D.F (2008). "Architects' perspectives on construction waste reduction by design". *Waste Management*, Vol. 28, No. 7, pp. 1147-1158.

Polat, G. and Ballard, G. (2004). "Waste in Turkish construction: need for lean construction techniques". *In Proceedings of the 12th Annual Conference of the International Group for Lean Construction IGLC-12, August, Denmark*, pp. 488-501

Ponnada, M.R., and Kameswari, P. (2015). "Construction and demolition waste management - a review". *Safety*, Vol. 84, pp.19-46.

Tam, V.W., and Hao, J. J. (2016). "Attitudes towards recycling on construction sites". *In Proceedings of the Institution of Civil Engineers-Waste and Resource Management*, Vol. 169, No. 3, pp. 131-136

Wahab, A.B., and Lawal, A.F. (2011). "An evaluation of waste control measures in construction industry in Nigeria". *Journal of Environmental Science and Technology*, Vol. 5, No. 3, pp. 246-254

Wahi, N., Joseph, C., Tawie, R., and Ikau, R. (2016). "Critical Review on Construction Waste Control Techniques: Legislative and Waste Management Perspective". *Procedia-Social and Behavioral Sciences*, Vol. 224, pp. 276-283.

Wang, J.Y., Kang, X.P., and Wing-Yan Tam, V. (2008). "An investigation of construction wastes: an empirical study in Shenzhen". *Journal of Engineering, Design and Technology*, Vol. 6, No. 3, pp. 227-236.

Won, J., Cheng, J.C., and Lee, G. (2016). "Quantification of construction waste prevented by BIM-based design validation: Case studies in South Korea". *Waste Management*, Vol. 49, pp. 170-180.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Safetycoin
An ultimate Reward-based Blockchain deployment for road user safety

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Abstract

The provision of road safety is a challenging issue faced by the global community. Taking up the road safety an urgent call, this research aims at utilizing innovative technologies led by Blockchain to address the complex mix of road safety variants. The research is aimed to enhance driver's behavior on road using risk/ reward strategy. Using 5th generation IT technology "the Blockchain", an effort is initiated to collect, analyze and justifiably reward safety drive behavior in a completely transparent environment. The novel feature of the research is the use of Blockchain to monetize real time human effort into a kind of crypto-token (SafetyCoin). The core of the research is the provision of risk/reward based sustained learning environment with the highest degree of transparency and minimum of human intervention. The provision of such a transparent environment is the challenge met using Blockchain deployment in real time for first time in the history of Blockchain. In the subject case, Ethereum, Proof of Stake (PoS) consensus algorithm is adopted to ledger real time human efforts towards safety drive. If fully developed and deployed in spirit, it would prove to a milestone in improving human behaviors in any respect through reward-based intervention techniques.

Keywords: Blockchain deployment, Road safety, driver's behavior, reward-based intervention, Ethereum

1. Introduction

United Nations Road Safety Strategy Report 2018 (UN, 2018), declares that about 1.3 million people yearly die from road accidents, which is more than 3000 deaths daily. It further exerts over 35 million people suffer from sustained injuries from roads which leads to an important cause of worldwide permanent disabilities. In Pakistan, the situation is not much different, WHO reports (WHO, 2018) a fatality figure exceeding 30,000 people yearly with an alarming number of 50,000 plus permanent disabilities. Road accidents are the fifth leading cause of deaths worldwide (UN, 2018).

Road safety at its core is a mix of inter-related complex factors of which drivers' risky behavior is the most complex element. Training an on-road driver is not a simpler task anymore specifically under a stressed environment which includes externalities like on road pollution, unexpected delays, traffic jams, and misconduct of other drivers (Shinar, 1998). Curbing the risky driving attitude needs not only improving drivers' attitude towards collective safety but also needs a sustained corrective environment which eventually might shift his psychic priorities towards safety concerns.
This research aims at utilizing a target-oriented gamification on a broader scale on top of P2P Blockchain infrastructure. At the core of the model lies a risk/ reward strategy as an incentive for the drivers to drive

safe. The research falls in the domain of applied technology and its conceptual framework consists of, but not limited to, the following:

1.1 Driver-Vehicle Interaction

The vehicle and the driver are two variants in the road safety paradigm with the roads as third. The most critical element is human error and accounts for a 90% share in a road accident. Hence the primary part of the research is establishing a driver vehicle partnership through the installation of the in-cabin comprehensive universal sensing system to monitor key aspects of driver's behavior towards traffic safety.

1.2 Blockchain Implementation and the Gamification

The processed output from sensors would be treated as input for the state variant in the Blockchain. It is the originality aspect of this research that the first time since its inception, the real-time human efforts (safe driver behavior) are treated as underlaying asset value of the token to be produced during mining. Yet another distinct feature is the token distribution, the drivers with higher standards of safer drives have higher stakes. Gamification has been utilized for training in several fields. As far as traffic safety, the same has led towards a more responsible driver's behavior. Such capacity building exercises are limited to a few big companies either in freight transport business or alike. The key parameter of gamification is about keeping the driver in a predefined, transparent digital environment with clear targets of winning or losing at the end of the drive call. Targets like speeding, braking, and lane changing per specified run, are achieved for reward and vice versa. Continuous exposure to such an environment would result in sustained learning towards traffic safety.

2. Conceptual Frameworks

Theoretical and conceptual frameworks along with the research methodology for all listed parts are detailed in short as under. The following concepts shall be utilized to put the proposed model up and running. The model is conceptualized initially to realize the null hypothesis (formulated research question), however, if implemented could prove itself a ground breaking technology on e-governance/ smart governance domains. Different phases of the research could be summarized as follows:

2.1 Sensors Installation and Data Conversion

Different kinds of sensors shall be installed in the car to monitor driver's behavior under predefined parameters. The ultimate goal of this phase would be the experimental production of a single portable monitoring device, which could be used in majority of vehicles. Conversion of recorded signals into a meaningful usable data using writing purpose specific routines and establishing reliable data transfer protocols and modalities shall be exercised during this phase of the project.

2.2 Data Analytics and Blockchain Deployment

Synthesized data then be analyzed and shared with driver specific risk aversion strategies and the same shall be used as an input stimulus for the Blockchain network.

2.4 Blockchain Development

A permission less public Blockchain, Ethereum on top of proof of stakes (PoS) consensus shall be used to record and award driver's risk avert behaviors. The modalities of the model are detailed in upcoming relevant section.

3. Literature Review

Aggression and distraction are the types of risky driving behaviors responsible mainly for a road crash (Jessor R., 1997), the risky driving is not only the reckless and aggressive driving but also inattentive, distracted and fatigue. Lack of attention (distraction), illness and sleepiness are depicted major causes of accidents in some studies (Minoiu Enache, 2009). Studies show there are several manifestations of risky driving, of which the most frequently mentioned are speeding and tailgating (James, 2000), (Sarkar S, 2000), (Tasca, 2000), (Jessor R., 1997). Others may include cutting in front other cars, weaving in and out of traffic, or running red lights (Shinar, 1998), (Sarkar S, 2000), (Lajunen T., 2001), (Tasca, 2000).

In a study, Tronsmoen, depicts that the amount of formal driver training was negatively associated with the respondents' evaluation of their driving skills. The results also showed that attitudes as well as self-assessment of driving ability were significantly associated with self-reported risk behavior. Young novice drivers' crash involvement seems strongly associated with self-proclaimed driving skills than safety attitudes (Tronsmoen, 2010). In another study, Taubman, finds parents' involvement in the intervention, either by feedback or by training, led to lower risky driving events rate of young drivers compared to the control group (Orit Taubman, 2015). It also suggests that the parents' and especially fathers' personality is associated with teens' risky driving.

Common approaches to curb the consequences of risky driving put enforcement of traffic laws at the top of the tooling list. Besides having several positive impacts on driver's behavior as mentioned in (Stenojevic P, 2013), such preventive measures because of its penalization nature, it may be source of excessive stress on driver's behavior. This is the most adopted approach to curb risky driving practices although it will not stop the risky driving completely. The results of the study by (Stenojevic P, 2013) indicates that the lack of enforcement affects almost every type of risky behavior. Drivers in less traffic enforcement regions drive faster, exceed speed limits more frequently, use seat belts less frequently, drive after exceeding the legal limit for alcohol more often, commit aggressive and ordinary violations more frequently and are generally involved in more risky situations (Stenojevic P, 2013).

Another approach may be to identify the risky drivers in traffic stream and inform in the immediate vicinity, and rest of the drivers amend themselves accordingly. In order to identify the different types of risky driving behavior, first of all the hazards which constitute a risk must be identified (Hoyos, 1988). These hazards can be present as fixed, stationary, or mobile objects in the driver's vicinity. It could only be possible in a complete vehicle to vehicle (v2v) communication infrastructure (Brown ID, 1988). Access infrastructure, such as Wi-Fi access points and cellular base stations (BSs), plays a vital role in providing pervasive Internet services to vehicles (Ning Lu, 2013). In their study (Ning Lu, 2013) discussed a variety of technologies which could be part of anticipated v2v and (vehicle to infrastructure (v2i) communication networks considering the capacity-cost tradeoffs for vehicular access networks of wireless access infrastructure. However, the major challenges posed is the automation transformation regime from truly analog legacy technology base to a complete digitized v2v environment.

Autonomous Vehicles (AVs) have the potential to alter traffic safety paradigm by averting deadly crashes, serving critical mobility to the elderly and disabled, increasing road capacity, saving fuel, and lowering emissions. Autonomous vehicles have the potential to dramatically reduce crashes. Yet, the proliferation of autonomous vehicles is far from guaranteed (Kockelman, 2006). High initial costs are critical barrier to the large-scale production and the mass consumer availability (Kokkelman, 2013). Potential benefits are of course, substantial but significant barriers to full implementation and mass-market penetration are still the challenges beyond comprehension. A framework for AV liability is almost absent, hence, creating uncertainty in event of a crash. Security concerns should also be given stress to be examined from a regulatory standpoint to protect privacy issues against data uses (Kockelman, 2006).

Our approach addresses curbing of the risky driving behavior using peer pressures in addition to reward incentives in a completely transparent and sustained, repetitive environment. It defines our research question as under:

3.1 Null Hypothesis

"A goal defined repetitive training in a truly transparent environment, where efforts to enhance safety driving behavior are accurately measured and appropriately rewarded, could be an effective tool to curb risky driving practices."

The critical question at this point is the creation of such an environment where drivers' efforts for risk aversion are accurately measured, analyzed and appropriately rewarded either in the form of reduced peer and social pressures or financial rewards of some kind. Fortunately, the innovations on the technological fronts have made it possible to create such an environment or at least could be given a try. The proposed model provides a mean and enhance the driver's risky behavior in real-time on-road safety domain using two very distinct but partner technologies namely, the sensor technology and the Blockchain technology. The research primarily be conducted in two phases, firstly to devise a system to accurately monitor driver's behavior in real time, and secondly to analyze, share and appropriately reward it.

3.2 Automobile Sensors and Sensing Technology

Sensors are essential components of automotive electronic control systems. Sensors are defined as the devices that transform (or transduce) physical quantities called measurands into electrical output signals that serve as inputs for control systems (Norton, 1989). The need for sensors is ever evolving with exponentially widening scope of usage. For example, in engine control applications, the number of sensors used will increase from approximately ten in 1995, to more than thirty in 2010, as predicted in (Powers, 1996). A milestone in sensor technology is the micro electro-mechanical systems MEMS development. MEMS came on scene in 1981, continued in the early 1990s with accelerometers to detect crash events for air bag safety systems and in later years had further developed with angular-rate inertial sensors for vehicle-stability chassis systems (Grace, 1999). Important features of the MEMS are the utilization of What the economy of batch processing along with miniaturization and integration of on-chip electronic intelligence (Sparks, 1998). MEMS provides an economical and efficient mean of high-performance sensors available for automotive applications (Sparks, 1998).

Table 1: System Applications for Automotive Sensors

As shown in Table, the three major areas of systems application for automotive sensors are powertrain, chassis, and body. In the present classification, the sensors not in the powertrain or chassis are included as a body systems application. It also identifies the main control functions of each area of application and the elements of the vehicle that are typically involved. The powertrain applications for sensors, can be thought of as the "1st Wave" of increased use of automotive sensors followed by the chassis applications as "2nd Wave" and body applications might be termed as the "3rd Wave (Fleming, 2001).

Area of System Application	**Elements Involved**
Powertrain Energy use, Drivability Performance	Engine, Transmission Onboard Diagnostics **(OBD)**
Chassis Vehicle handling, Safety	Steering, Suspension Braking, Stability
Body Occupant's needs	Safety, Security Comfort, Convenience, Information

Today's modern automobiles have a variety of sensors. Body sensors include a diverse range consisting of, but not limited to crash-sensing accelerometers, ultrasonic near-obstacle sensors, infrared thermal imaging, millimeter-wave radar, ambient-air electrochemical gas sensors (D. Teegarden, 1998). New types of sensors, currently installed in body systems applications, include lateral lane-departure warning, the ultrasonic-array reversing aid, and infrared-thermal imaging night-vision sensors (Murphy, 2000). The list may include variety of new born sensors like, the mass air flow sensor, the engine speed sensor, oxygen sensor, manifold absolute pressure sensor, spark knock sensor, fuel temperature sensor, voltage sensor and several others (Azuma, 2018).

The Onboard Diagnostic-2 OBD2, is a standard system available in all types of vehicles manufactured after 1996 (Azuma, 2018). The sensor monitors the functioning of the vehicle and if possible, regulates its operation. An OBD2 system collects data from a plenty of components which may include the engine RPM, crankshaft position, run speed, air temperature, etc. An OBD2 scanner is not necessarily, compatible with all the makes and models running on roads in Pakistan. However, OBD2 port quite possibly be used to collect data regarding safety parameters of the drive. There are certain safety parameters which are not possible to obtain via available sensing setup. Hence, we would need to either transform in place sensing system of to install new updated format for safety specific parameters.

4. The Blockchain

Blockchain is a 5th generation internet technology and presumably would be leading next technological revolution (Tapscott, 2016). It provides an infrastructure for the applications to record digitally signed transactions in the form of a digital ledger, propagated on a P2P network. Digital signatures are meant to ensure authenticity and the highest level of integrity of the data. Within the Blockchain paradigm, mining is the transaction validation process that runs on a certain cryptographic algorithm to ascertain system transparency. By design, a Blockchain is resistant to modification of the data (Wikipedia, n.d.). The fundamental elements of the Blockchain infrastructure (thereafter referred to as Blockchain) could be summarized as follows:

1- **Network protocol**: A Peer to peer (P2P) computer network
2- **Cryptocurrency:** A substance (a digital form of an asset)
3- **Transaction**: A cryptographic transaction mechanism
4- **Mining protocol:** A fool proof transaction validation mechanism
5- **Propagation:** A mechanism to propagate validated transaction record on network

4.1 A Peep into Blockchain

The typical building blocks of a Blockchain network are invariably identical in different versions. Such as, it needs a P2P network protocol and a cryptographic mechanism to secure transactions on a P2P network. It needs as well, a powerful, flawless mechanism for transaction validation in real time with an un-immutable character. This irreplaceable functioning provides a fundamental Blockchain infrastructure, on top of which unlimited purpose specific applications are built to serve its own domain. A summary is reflected in Table-2 and Table-3.

Table- 2: Characteristic Parameters of Blockchain

Parameter	Public	Consortium	Private
Validation Consensus	All miners	Selected nodes	Organization
Read permission	Public	Public/ restricted	Public/ restricted
Immutability	Impossible	Possible	Possible
Efficiency	Low	High	High
Centralization	No	Partial	Yes

Mining is the transaction validation through a consensus protocol (algorithm). It is the segment where strength, utility, and purpose a Blockchain is determined. for instance, proof of work (PoW) algorithm demands a huge power usage, depending on the miner location, an average of electric power of value US$ 4000 is required to generate a bitcoin. Moreover, the transaction process is slow along with a lesser number of one block transactions. On the contrary, proof of stakes (PoS) algorithm requires much lesser energy with an efficient transaction rate with some additional usage features like smart contracts. The question, how mining is done in either of the protocol shall be addressed later in the subsequent section.

Table- 3: Parameters of Blockchain Consensus Protocols

Consensus Protocol	PoW	PoS	PBFT	DPOS	Ripple
P2P Protocol	Open	Open	Permissioned	Open	Open
Energy Saving	No	Partial	Yes	Partial	Yes
Tolerance	<25% power	<51% stake	<33.3% replicas	<51% validators	<20% faulty nodes
Example	Bitcoin	Peercoin, Ether	Hyper ledger	Bit shares	Ripple

Token is the output of the whole laborious transaction validation exercise and visually is a meaningless cryptographic unique hash of specified characters. This uniqueness and value addition of the data secured in the background, make it an asset. This unique number is traded as an online monetary substitute (cryptocurrency). At this point in time, more than a hundred cryptocurrencies are roaming in the internet world. Some like, bitcoin, ether, ripple, Litecoin, altcoin, etc. are popular than others.

5. How Blockchain fits our model

As had already been outlined, our goal is the construct of a road safety real time “gamification like” model, through which drivers are incentivized to join a sustained environment developed for driving behavior improvements towards safety concerns. At this stage, it becomes important to define driver safety parameters. The key parameters might include, but not limited to, the following:

Seat Belt	Seat belt messages and alarming
Fixtures	Fixtures auto settings or messaging
Speed	Over speeding events per unit length of run
Braking	Sharp brakes per unit length of run
Lane Changes	Abrupt lane changing events per unit length of run
Distractions	Driver’s distraction, including mobile usage, texting and drowsiness

6. Research Methodology

6.1 Driver-Vehicle Interaction

Conventionally drivers do know their cars but cars don’t know their drivers. For a safe drive, both these variables should know each other to build a partnership. The task could be achieved by applying different aspects of sensor technology. In fact, a number of technologies (such as OBD2) already exist and are being applied to new models of the vehicles. Our main issue would be to apply the knowledge set for the vehicles running in the majority on domestic roads. It would bring enormous challenges to the subject research.

To achieve the said interaction, the vehicle would be equipped to recognize its driver via his license, finger prints or facial recognition. A simultaneous synchronization with the vehicle sensing apparatus and the mobile application on driver's mobile shall be established to ensure data streaming. As it wouldn't be a V2V communication, extra DSRC arrangements shall not be required. Vehicle's ignition shall be synchronized with the confirmation parameters to establish the following:

6.2 Blockchain Deployment

The spirit of the subject research lies in part-2, where the data comprised of driver's behavior shall be organized, analyzed and processed for the Blockchain input segment. Here it is solely important to understand that it is the first time when human efforts are directly be fed as an input for the Blockchain algorithm which would make the tokens generated a real-world asset and hence would initiate an increase in value and subsequently increase in demand. This is the asset value of the token which might make people join the "Drive-safe" network. Being sensible enough, we would not try to dig the well twice and try to amend and apply available technologies and developments in our domestic conditions (for domestically in-use vehicles).

Ethereum is a distributed public block chain network that focuses on running programming code of any decentralized application. More simply, it is a platform for sharing information across the globe that cannot be manipulated or changed (Blockchain.com, n.d.). Ether is a decentralized digital currency, also known as ETH on Ethereum Blockchain. In addition to being a tradeable crypto, ether powers the Ethereum network by paying for transaction fees and computational services. Ether is paving the way for a more intelligent financial platform.

7. Expected Outcomes

Road safety is a major public health/ law and order issue globally. According to estimates, human error accounts for a 90% cause of on road accidents. Human error roots deeply in driver's perception of safety drive. A great deal of research is focused on how to intervene driver's on road behavior towards positive road safety behavior. Although stricter law enforcement based on penalizing strategies, could improve the situation for but only at a limited scope. This research is focused on devising a digital solution for a sustainable, result oriented, and reward-based safety drive environment. The driver shall be continuously and voluntarily exposed to transparent digitally gamified driving environment where its efforts to safe drive shall be monetized and rewarded.

It is intended to introduce "Safecoin" as the token (ICO) for our new Safety Blockchain, running on top Ethereum Blockchain infrastructure. It shall be a publicly accessed Blockchain with PoS (Proof of stack) consensus algorithm. The primary reason to adopt Ethereum is its scalability, power efficiency and flexibility for adoption.

The revolutionary innovation of this research is twofold:

- It would be first of its kind of Blockchain deployment where real time human efforts shall be traded in the form of 'Safecoin", which would induct an enormous intrinsic asset value.
- The process of "Safecoin" generation would not differ from the basic consensus routines, however, the driver with higher safety scores shall have higher stacks, and would have higher chances of being awarded.

If deployed in its spirit, the Safety Blockchain might create new horizons in award base smart governance efforts as it would be applied in almost any domain of social life.

8. Bibliography and References

Azuma, T. (2018, June 21). *Types of Car Sensors Used in Automobile Engine*. Retrieved from carfromjapan.com: https://carfromjapan.com/article/car-maintenance/types-of-sensors-used-in-automobile-engine/

Brown ID, G. J. (1988). Risk perception and decision making during the transition during the transition between novice and experienced driver status. *Ergonomics 31(4)*, 585-97.

D. Teegarden, G. L. (1998). How to model and simulate micro gyroscope systems. *IEEE Spectrum*, 67-75.

Grace, R. (1999). Application opportunities of MEMS/MST in the automotive market. *Proc. Vehicle Displays Microsens*, 25–31.

Hoyos, C. (1988). Mental Load and risk in traffic behavior. *Ergonomics 31*, 571-84.

ibin Zheng, S. X. (2017). An Overview of Blockchain Technology: Past, present and future. *2017 IEEE 6th International Congress on Big Data*, (pp. 557-564). Hon Kong.

James, L. a. (2000). Road Rage and Aggressive Driving: Steering clear of highway warfare. *Amherest.* NY: Prometheus.

Jessor R., T. M. (1997). Predicting Development change in risky driving: the transition to young adulthood. *Applied Development Sciences*, 4-16.

Kockelman, K. S. (2006). Road pricing simulations: Traffic, land use and welfare impacts on Austin, Texas. *Transportation Planning and Technology 29(1)*, 1-23.

Kokkelman, K. m. (2013). *Preparing a nation for autonomous vehicles: opportunities, barriers and policy*. Texas, US: Eno Center for Transportation.

Lajunen T., P. D. (2001). Are aggressive people aggressive drivers? A study of the relationship between self-reported general aggressiveness, driver anger and aggressive driving. *Accident analysis and prevention 33*, 243-55.

Ministry of Communications Pakistan, M. P. (2019). *National Transport Safety Policy, Pakistan 2018-2030.* Islamabad: MoC Pakistan.

Minoiu Enache, N. N. (2009). Driver steering assistance for lane driver departure avoidance. *Control Engineering Practices 17(6)*, 542-651.

Murphy, T. (2000). Rear vision made simple," , pp. 57–58,. *Ward's Auto World*, 57-58.

Ning Lu, N. Z. (2013). *IEEE Truncation on Intelligence*. Retrieved from https://ieeexplore.ieee.org/abstract/document/6516010

Norton, H. (1989). Transducer fundamentals. In N. Englewood Cliffs, *Handbook of Transducers* (p. Ch. 2.). Prentice Hall.

Orit Taubman, B. A. (2015). Parents and peer contribution to risky driving of male drivers. *Accident analysis and prevention Volume 78, Issue 1*, 81-86.

Powers, M. B. (1996). The role of electronic controls for future automotive mechatronic systems. *IEEE/ASME Trans. Mechatronics vol.1*, 80–88.

Sarkar S, M. A. (2000). Spatial and temporal analysis of the variations in aggressive driving and road rage behavior observed and reported on San Diego freeways. *Transportation Research Record 1724*, 7-13.

Shinar, D. (1998). Aggressive driving: the contribution of the driver and the situation. *Transportation research Part F: Traffic Psychology and behavior 1(2)*, 137-160.

Sparks, D. E. (1998). Application of MEMS technology in automotive sensors and actuators. *IEEE, vol. 86*, 1747–1755.

Stenojevic P, J. D. (2013). Influence of traffic enforcement on attitude and behavior of drivers. *Accident Analysis Prevention*, 29-38.

Tasca. (2000). A review of the literature on aggressive driving research. *Global Web Conference on Aggressive Driving.*

Tronsmoen, T. (2010). Association between driving training, determinants of risky driving behavior and crash involvement. *Elsevier Science Digest Volume 4 Issue 1*, 35 45.

UN, U. (2018). *Road Safety Strategy Report.* Geneva: United Nations.

WHO (2018). *WHO Report on Road Safety.* Washington: World Health Organization.

Wikipedia. (n.d.). *Wikipedia*.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Impact on Local Scouring Around Bridge Pier by Submerged Trapezoidal Broad Crested Weir

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Abstract
Scouring around the bridge pier is the crucial factor that causes the failure of bridge piers. During the scouring process, removal of sediments take place around the bridge piers. As a result, the exposure of foundation occurs which ultimately leads to bridge failure. Different technique were used to minimize the scouring effect around bridge pier. In this experimental study, the interference of local scouring between single circular pier diameter (D) of 6cm and downstream submerged trapezoidal broad crested weir (TBCW), height of weir (Z) is 5cm with different separation distance (S) D, 2D, 3D, 4D. The experimental study was carried out in flume present at Hydraulics Engineering Laboratory, University of Engineering and Technology Taxila, using uniform bed material, flow depth‒flowing under the clear water condition. The scouring depth generated at upstream face of the pier, in presence of the TBCW, was compared with the local scouring without TBCW‒at downstream. Based upon the experimental results shows that the scour depth at circular pier can be reduced up to 41.3% by incorporating the submerged TBCW‒at downstream‒at S=D as compared to the single pier without the TBCW. The reduction in scouring decreases with increase in separation distances between pier and weir.

Key Words
Single circular Bridge Pier, Trapezoidal Broad crested weir (TBCW), Separation Distance, Local Scouring Mechanism

6. Introduction

Scouring has been identified as an extensive and critical factor in bridge pier design. Exaggerated scouring around the bridge piers can leading failure of a bridge structure. Scouring is a major failure cause of bridges, and have a huge maintenance and construction cost (FDOT, 2005). 60% of bridges failed due to channel bed scouring and volatility of channel bed, out of 823 in USA since 1950, approximately 50 bridges failed annually according to US Federal Highway Association (Shirole & Holt, 1991). In Pakistan During 2010 floods, more than 278 bridges were collapsed only in KPK Province. (Ferderal Flood Commission, 2010). Figure. 1 shows the scour effect on the bridge piers. However, during the flood flows conditions and similarly man made alternate increased the sediment carrying capacity of the river then general scour also occur (Breusers and Raudkivi 1991). Local scouring and general scouring both may cause structure damage or failure. Conventionally submerged weirs (e.g check dam, bed sill) are constructed at the downstream of bridge pier as a countermeasure of general scouring. Bed sill can be used as countermeasure for the local scouring at the bridge pier under the clear water scour condition by using the bed sill immediately at the downstream of the pier the scour depth at the pier can be reduced by up to 26 %. (Grimaldi et al., 2009a). This reduction can be increased by the proper combination of bed sill with other proper flow altering countermeasure. (Grimaldi et al., 2009b)(Gaudio et al., 2012); Due to the presence of debris stockpiling, reduction in the pier scouring can be reduced (Pagliara et al., 2010) The local scouring also occur due to presence of a submerged weir in a river. In the past decades local scour at downstream of weir like structures has been broadly investigated. (Gaudio et al., 2000)(Mario A. Lenzi et al., 2002)(Mario A. Lenzi et al., 2003a)(M. A. Lenzi et al., 2003b)(Marion et al., 2004)(Meftah & Mossa, 2006)(Marion et al., 2006)(Pagliara & Kurdistani, 2013)(Guan et al., 2016)(Wang et al., 2018a)(Wang, Melville, Guan, et al., 2018b).

(a) (b)

Figure 1: (a) Bridge Collapsed During 2010 Floods in Pakistan (FFC Report, 2010) (b) Scour Effect on Bridge in Harrow River

In this experimental study, investigate the effect on single circular bridge pier scouring after installation of downstream submerged trapezoidal broad crested weir (TBCW), under the clear-water flow condition, different type of experiment was conducted, first place the pier immediately near to the weir then increase separation distance from weir. The effect of scouring was measured for each case and then compared the scour results.

7. Materials And Methodology

All the experiment were performed in the Water resources & Hydraulics Engineering Laboratory of Department of Civil Engineering, University of Engineering and Technology, Taxila. The channel had specifications of 20 m length, 0.96 m wide and 0.75 m deep glass-sided flume with an adjustable tailgate at the channel-end to regulate the flow depth used for the experiment. Through the pump, discharge was supplied to the tank then inter to the main channel by aligned honeycomb diffuser in the direction of flow for smooth and uniformly distribution of flow cross-wise. At the tail of channel water enter in the sediment tank after filling and trapping the flowed sediments, the flow discharges in the main channel. Plan view of the Laboratory channel and experimental setup shown in Figure 2. Also pictorial representation of experimental setup shown in Figure 3.

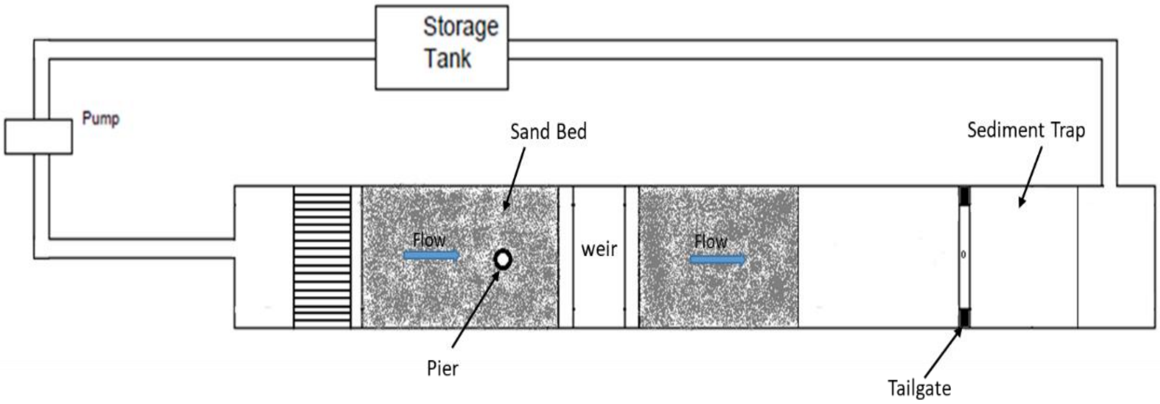

Figure 2: Plan View of Channel

A 9m long, 0.20m deep and .96m wide false bottom of uniform, medium size (d50=.51mm) bed material and pier base was introduce, sand size considered in this study stood in compliance to the condition of D/d50 > 50 in order to dominate the sediment size effect on the scour evolution process guaranteeing non–ripple-forming sand(Raudkivi & Ettema, 1983)(Ettema, Melville, & Barkdoll, 1998). The diameter of the pier was not more than 10% of the channel width to prevent the effect of walls on scouring (Chiew and Melville, 1987), therefore single circular pier modelled by 0.06m diameter PVC material. The trapezoidal broad crested weir (TBCW) modelled by wood material, weir width equal to the full width of flume, crest length was 50cm, the weir was 5cm above the flat bed, the sides slopes of the TBCW was 1V:2H ,which is more stable and seepage control.(Fritzi & Hager, 1998). The dimension of TBCW was within the limits(Sturm, 2001)(Henderson 1966) (Chanson, 2004).

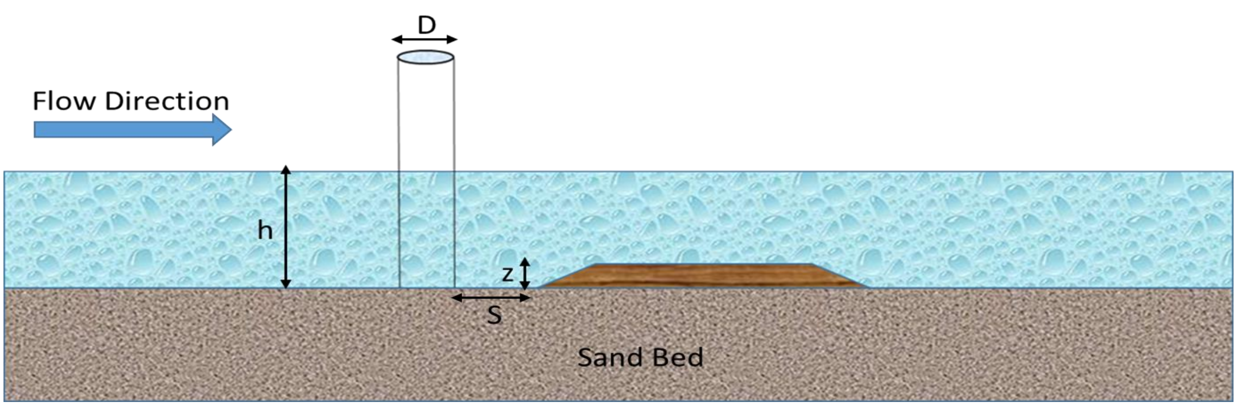

Figure 3: Pictorial Representation of Experimental Setup.

Single circular pier was placed in the center of the flume, then at the downstream of the pier TBCW was installed. In start the pier was placed near to the weir, separation distance from the TBCW was D, then increase the separation distance (S) with respect to the diameter of pier as D, 2D, 3D and 4D. The sand bed was carefully levelled to flat bed with upstream and downstream flume bed, before performing the every experiment. The discharge was then allowed to enter flume gradually and attained the flow depth and velocity became low with the help of tailgate channel. The discharge was set and flow depth was regulated manually to maintain constant and then stabilized the discharge according to conditions. When the depth of water achieved then started the experiment time. When the experiment completed, turned off the pump and drain the flume slowly.

After the completion of drainage, point gauge was used to measure the scouring around the pier having an accuracy of ± 0.5 mm. The experimental minimum time, allowed to run was 2 hour for each experiment. The suitable equilibrium time is 2 hour for the experiment (Karimi et al., 2017). Therefore average time taken for each experiment was 2 hour. After completion of the above procedure, measured the geometrical dimensions of the scoured hole.

8. Result And Discussion

All the experiment was performed in clear water flow condition, which means that the mean velocity of the flow was less or equal to the threshold velocity or critical velocity (Vc), in clear-water scouring condition maximum scouring occurs at the threshold peak. (Melville and Chiew, 1999) Where 'h' is the flow depth, 'T' is the time in hour, 'Q' is the discharge, 'S' is the separation distance from the downstream submerged TBCW, D_{s_f} is the maximum scour at the upstream face of pier. Table: 1 show the hydraulic parameter of present experimental study.

Table 1: Experimental Condition and Result

Case	Q (cumecs)	T (hr)	h (m)	S (m)	Ds_f (cm)	Ds_f/D
1	0.035	2	0.15	without Weir	4.6	0.767
2	0.035	2	0.15	1D	2.7	0.450
3	0.035	2	0.15	2D	3.1	0.517
4	0.035	2	0.15	3D	3.6	0.600
5	0.035	2	0.15	4D	4.0	0.667

In first case scouring around the single pier was measured without presence of downstream submerged TBCW, then installed the submerged TBCW at downstream of pier and varies the separation distance (S) between the pier and TBCW with respect to pier diameter as D, 2D, 3D, 4D and measure the scouring around the pier and compare the result of single circular pier scouring with or without presence of downstream submerged TBCW. During experiment it was observed that maximum reduction of scouring up to 41.3% at S=D, by increasing the separation distance between the pier and weir the reduction in scouring around the pier reduced.

Figure. 4 show the maximum scour depth near the pier with reference to pier diameter at different separation distance. Figure. 5 show the percentage reduction in scouring at various separation distance from TBCW as compared to the single circular bridge pier scouring without present of downstream submerged TBCW. 3D view and Contour map of scour hole of the pier without presence of downstream submerged TBCW shown in Figure 6.

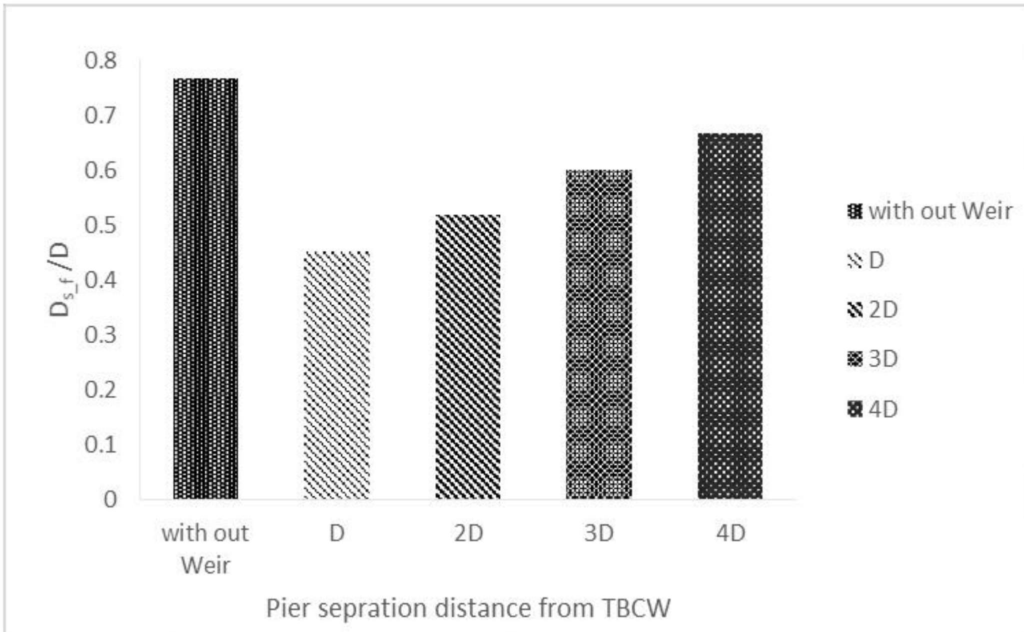

Figure 4: Scour Depth near the Pier

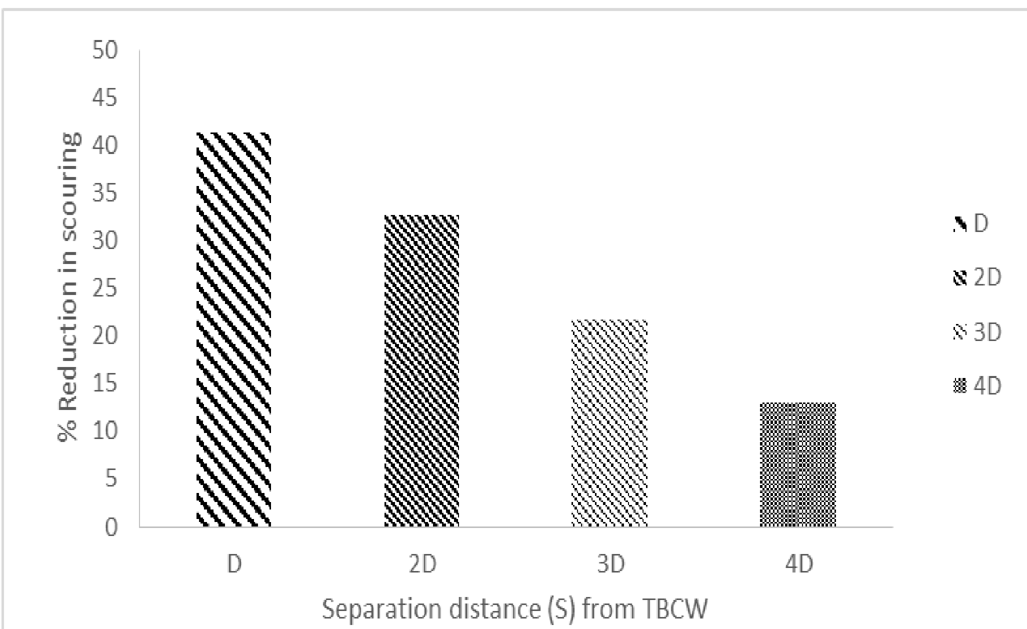

Figure 5: Percentage Reduction in Scouring

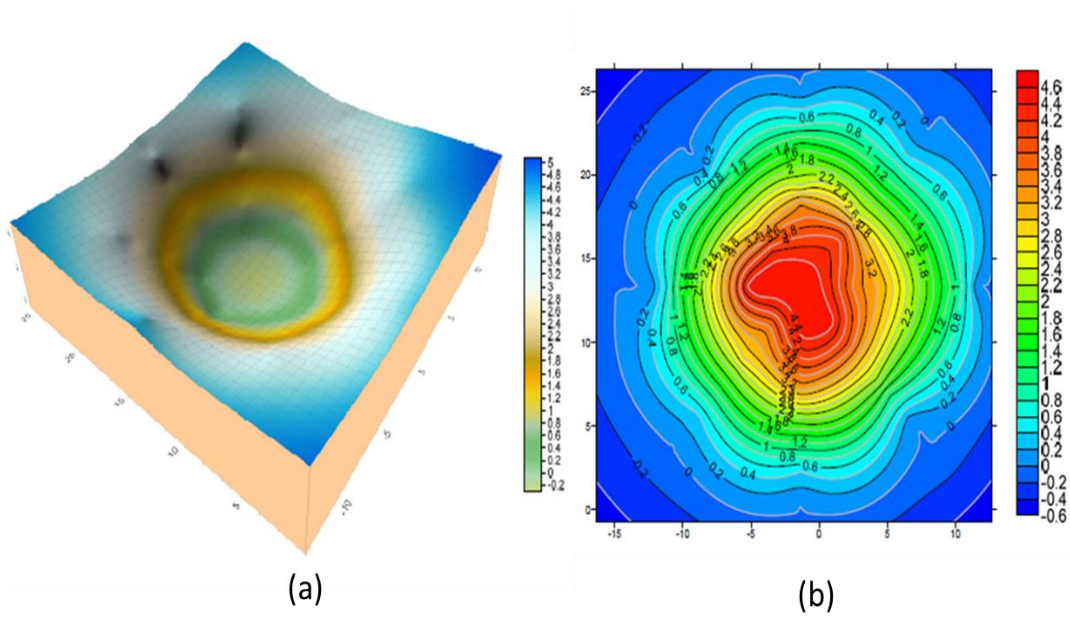

Figure 6: a) 3D View b) Contour Map

Figure. 7 shows the relationship between non-dimensional scour depths denoted by D_{s_f}/D and the non-dimensional longitudinal distance of scour hole from upstream face of the pier, denoted by L/D.

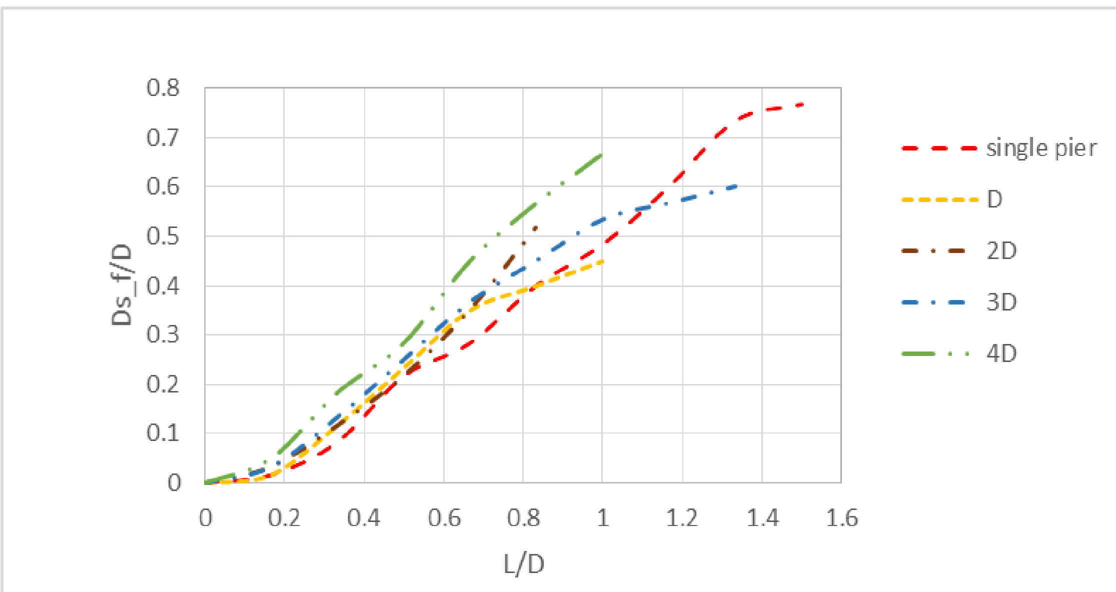

Figure 7: Longitudinal Scour Profile

9. Conclusions

Experimental study, investigate the scour pattern around single circular pier with or without presence of downstream submerged TBCW and by varying the separation distance of pier from the weir. Conclusion have been drawn that when pier was at the separation distance S=D from the TBCW, in this case minimum scouring were observed around the pier, reduction of scouring up to 41.3%, then by increasing the separation distance from D to 4D between the pier and weir it observed that the reduction in scouring reduced. The reduction in pier scouring up to 13%was observed when the separation distance S=4D. However, the depth of scour hole was increased with increasing the distance from the weir.

10. References

Ettema, Melville, & Barkdoll, (1998). *Scale effect in pier-scour experiments. Journal of Hydraulic Engineering, Vol. 124, No.6, June,1998*

FDOT. (2005). *State of Florida Department of Transportation Bridge Scour Manual*. *May*.

Ferderal Flood Commission. (2010). Annual Flood Report Office of the Chief Engineering Adviser &. *Ministry of Water & Power*, 1–51.

For, C., At, C., By, I., Melville, B. W., & Chiew, Y. (1999). *Cale for*. *May 2010*, 59–65.

Fritzi, H. M., & Hager, W. H. (1998). HYDRAULICS OF EMBANKMENT WEIRS By Hermann M. Fritzi and Willi H. Hager,z Fellow, ASCE. *Manager*, *124*(September), 963–971.

Gaudio, R., Marion, A., & Bovolin, V. (2000). Morphological effects of bed sills in degrading rivers. *Journal of Hydraulic Research*, *38*(2), 89–96. https://doi.org/10.1080/00221680009498344

Gaudio, R., Tafarojnoruz, A., & Calomino, F. (2012). Combined flow-altering countermeasures against bridge pier scour. *Journal of Hydraulic Research*, *50*(1), 35–43. https://doi.org/10.1080/00221686.2011.649548

Grimaldi, C., Gaudio, R., Calomino, F., & Cardoso, A. H. (2009a). Control of scour at bridge piers by a downstream bed sill. *Journal of Hydraulic Engineering*, *135*(1), 13–21. https://doi.org/10.1061/(ASCE)0733-9429(2009)135:1(13)

Grimaldi, C., Gaudio, R., Calomino, F., & Cardoso, A. H. (2009b). Countermeasures against local scouring at bridge piers: Slot and combined system of slot and bed sill. *Journal of Hydraulic Engineering*, *135*(5), 425–431. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000035

Guan, D., Melville, B., & Friedrich, H. (2016). Local scour at submerged weirs in sand-bed channels. *Journal of Hydraulic Research*, *54*(2), 172–184. https://doi.org/10.1080/00221686.2015.1132275

Karimi, N., Heidarnejad, M., & Masjedi, A. (2017). Scour depth at inclined bridge piers along a straight path: A laboratory study. *Engineering Science and Technology, an International Journal*, *20*(4), 1302–1307. https://doi.org/10.1016/j.jestch.2017.07.004

Lenzi, M. A., Marion, A., & Comiti, F. (2003). Local scouring at grade-control structures in alluvial mountain rivers. *Water Resources Research*, *39*(7). https://doi.org/10.1029/2002WR001815

Lenzi, Mario A., Marion, A., & Comiti, F. (2003). Interference processes on scouring at bed sills. *Earth Surface Processes and Landforms*, *28*(1), 99–110. https://doi.org/10.1002/esp.433

Lenzi, Mario A., Marion, A., Comiti, F., & Gaudio, R. (2002). Local scouring in low and high gradient streams at bed sills. *Journal of Hydraulic Research*, *40*(6), 731–739. https://doi.org/10.1080/00221680209499919

Marion, A., Lenzi, M. A., & Comiti, F. (2004). Effect of sill spacing and sediment size grading on scouring at grade-control structures. *Earth Surface Processes and Landforms*, *29*(8), 983–993. https://doi.org/10.1002/esp.1081

Marion, A., Tregnaghi, M., & Tait, S. (2006). Sediment supply and local scouring at bed sills in high-gradient streams. *Water Resources Research*, *42*(6), 1–11. https://doi.org/10.1029/2005WR004124

Meftah, M. Ben, & Mossa, M. (2006). Scour holes downstream of bed sills in low-gradient channels. *Journal of Hydraulic Research*, *44*(4), 497–509. https://doi.org/10.1080/00221686.2006.9521701

Chanson, H. (2004). “The hydraulics of open channel flow: An introduction”. 2d edition. Department of Civil Engineering, University of Queensland. Australia.

Pagliara, S., Carnacina, I., & Cigni, F. (2010). Sills and gabions as countermeasures at bridge pier in presence of debris accumulations. *Journal of Hydraulic Research*, *48*(6), 764–774. https://doi.org/10.1080/00221686.2010.528184

Pagliara, S., & Kurdistani, S. M. (2013). Scour downstream of cross-vane structures. *Journal of Hydro-Environment Research*, *7*(4), 236–242. https://doi.org/10.1016/j.jher.2013.02.002

Raudkivi, A. J., & Ettema, R. (1983). Clear-water scour at cylindrical piers. *Journal of Hydraulic*

Engineering, *109*(3), 338–350. https://doi.org/10.1061/(ASCE)0733-9429(1983)109:3(338)
Shirole, A. M., & Holt, R. C. (1991). Planning for a comprehensive bridge safety assurance program. In *Transport Research Record* (Vol. 1290, pp. 137–142).
Sturm, T. (2001). *Open hydraulics channel*. 493.
Wang, L., Melville, B. W., & Guan, D. (2018). Effects of upstream weir slope on local scour at submerged weirs. *Journal of Hydraulic Engineering*, *144*(3), 1–9. https://doi.org/10.1061/(ASCE)HY.1943-7900.0001431
Wang, L., Melville, B. W., Guan, D., & Whittaker, C. N. (2018). Local scour at downstream sloped submerged weirs. *Journal of Hydraulic Engineering*, *144*(8), 1–8. https://doi.org/10.1061/(ASCE)HY.1943-7900.0001492

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Investigating the Effects of Finite Length Rigid Emergent Vegetation on Inland Flood Attenuation

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Abstract
Due to extreme storm events, most of the areas in Pakistan are susceptible to large scale inundation which causes significant damages. The role of rigid emergent vegetation has been widely studied in the past for inland flood protection. Volume of Fluid (VOF) modelling is applied in the current study to simulate flow through vegetation in open channel. The affect of vegetation conditions of varying aspect ratio (vegetation length to width ratio) against constant vegetation density (number of vegetation elements occupying unit cross-sectional area) was investigated on mean flow structure. Channel domain was modelled in ANSYS workbench which was then exported to CFD code FLUENT for flow simulation. The simulation results were validated against existing experimental data to authenticate numerical model. At critical locations i.e. vegetation upstream and downstream, the velocity and water level variations for three distinct aspect ratios, were studied. The results show that by increasing aspect ratio from 1.01 to 2.40, there is corresponding increase in velocity reduction, backwater rise and water level drop behind vegetation. However, accelerated flow was observed inside the vacant space between vegetation edge and channel side walls. Thus, establishing optimum vegetation around rivers significantly affects the inland flood flow structure.

Keywords: Rigid emergent vegetation, VOF Modelling, aspect ratio, vegetation density, backwater rise

1. Introduction

Floods around the globe are considered as one of the most serious environmental threat. In the recent past Pakistan has witnessed one of the devastating floods in the history. Although floods are common throughout the country, but their nature varies with geography of area. Considering all the population struck by natural calamities, almost 90% of them are affected by floods. As per latest statistics mentioned in Annual Flood Report 2018, Pakistan has suffered cumulative financial loss of more than US$ 38.171 billion over the past 71 years. In 2010, Pakistan had faced one of the devastating floods in the history that cost colossal loss of economy and lives (Tariq and Van de Giesen, 2012). Therefore, keeping in view of the above facts it is obvious that flood is major problem which greatly effects the socio-economic development of a country. The extreme storm events result in high discharge which overflows the river in lowland areas and eventually causes flooding. The major characteristics associated with floods include high flow velocity and large inundation depth. Many researchers in the recent past have worked on development of cost and environment efficient approaches that include physical models, numerical models and analytical methodologies (Tanaka *et al*., 2007; Igarashi *et al*., 2018; Pasha and Tanaka, 2019). Generally, for a third world country like Pakistan, such approaches demand significant capital investment which make them difficult to execute. Presently, eco-friendly methods are adopted around the globe which not only effective in mitigating flood flows but also caters the degrading environmental situation. In this aspect, establishing vegetation in river floodplains can provide effective resistance to flow (Västilä *et al*., 2013). The vegetation cover is considered crucial as it enhances the catchment's storage capacity by increasing the lag time, decreasing the discharge and flood hazard (McDonald *et al*., 2016).

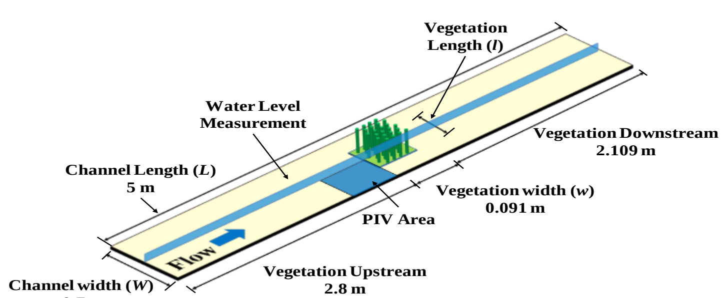
The effectiveness of vegetation for inland flood protection depends on various factors. Iimura and Tanaka (2013) studied the effects of aspect ratio on tsunami mitigation and concluded by numerical simulations that the tsunami force behind vegetation increased as the aspect ratio increased from 1 to 4. This was due to diffraction of tsunami waves by the forest (vegetation) edge. Moreover, the post-tsunami surveys in Japan showed that trees (vegetation) helped in reducing costal damages. Zaha *et al*. (2019) and Ahmed and Ghumman (2019) carried out experiments to study a hybrid defence system comprising of a forest, a moat (depression) and an embankment under unsteady flow conditions and found that hybrid defence system offers maximum resistance to flow. Pasha and Tanaka (2016) performed series of experiment to investigate role of finite length rigid emergent vegetation in trapping tsunami debris. Moreover, Volume of Fluid (VOF) modelling has been used in the past for simulating open channel flows, flow over weir crest and stepped spillway (Ramamurthy *et al*., 2007; Debnath, *et al*., 2008; Zhan *et al*., 2016). However, VOF modelling to investigate flow through vegetation of varying aspect ratio has not been widely studied in the past.

The current study aims at numerical investigation of flow behaviour through rigid emergent finite length vegetation of varying aspect ratio. This research provide information related to the design of bio-shield (vegetation) for inland flood protection.

2. Materials and Methods

2.1. Experimental setup

Pasha and Tanaka (2016) carried out experiments on series of vegetation conditions as shown in Table 1. All experiments were conducted in a water flume in Saitama University having constant bed slope of 1/2400, length of 5m, width of 0.7m and height of 0.5m. The initial water depth of 0.045m was taken against Froude number (dimensionless number which is ratio of inertial to gravitational forces) of 0.7. This was due to the reason that at many inland locations, flooded by tsunamis, the flow was sub critical and Froude number was determined between 0.7 and 1. Vegetation model was attached to the bottom of flume and placed 2.8m from inlet. The model was located carefully at the middle of channel width. The flow through vegetation model is shown in Fig. 1b. Fig. 1a shows schematic diagram of experimental setup. Velocity distribution in front of vegetation was measured with the help of Particle Image Velocimetry (PIV) (Laser Light Sheet: G200, high-speed digital CCD camera: KII, fps: 50–1000, flow analysing software: FlowExpert2D2C, Katokoken Co., Ltd.) in the transverse direction (Fig.1c).



(a)

(b) (c)

Fig. 1. Experimental Setup: (a) schematic diagram of experimental setup. (b) flow through sparse vegetation model. (c) PIV working setup

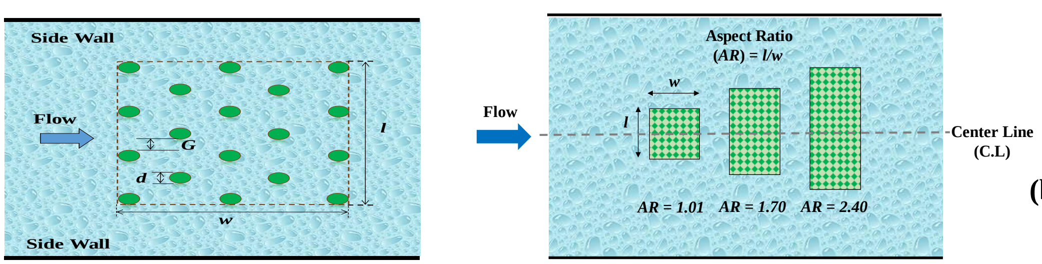
Table 1: Vegetation Cases (Pasha and Tanka, 2016)

Case No.	Aspect Ratio	Vegetation Spacing to Diameter Ratio	Vegetation Density (vegetation elements/cm^2)	Vegetation Type
	AR	***G/d***	***dn***	
1	1.01	2.13	0.24	Sparse
2	1.7			
3	2.4			

2.2. Vegetation conditions

The vegetation specie selected for modelling was pine tree having trunk diameter of 0.4m and average height of 15m. Considering mean diameter of pine tree, the scaling was selected as 1/100 according to which the vegetation elements were modelled by means of wooden cylinders having diameter of 0.004. Pasha and Tanaka, (2016) studied the effect of *G/d* ratio (where, *G* is the spacing between the cylinders in cross-stream direction and *d* is the diameter of cylinder) of various vegetation layouts on flow structure (Fig. 2a). Vegetation gap to diameter ratio (*G/d*) represents the density of vegetation cylinders which is kept constant i.e. 2.13, for current study. Vegetation density (*dn*), defined as number of cylinders occupying unit cross-sectional area, was computed for *G/d* ratio of 2.13 and found to be 0.24 cylinders/cm^2. Iimura and Tanaka (2013) studied that flow structure behind vegetation depends on aspect ratio (vegetation length to width ratio i.e. *l/w*). Three aspect ratios (*AR)* of 1.01, 1.70 and 2.40 were selected to model finite length inland vegetation patches Fig. 2b. The width of vegetation (*w*) was kept constant for all three cases.

(a)



(b)

Fig. 2. Schematic diagrams: (a) arrangement of vegetation cylinders inside a patch and definition of various parameters. (b) definition of aspect ratio (*AR*) and vegetation conditions involving *AR* of 1.01, 1.70 and 2.40.

2.3. VOF modelling and computational domain

Volume of fluid (VOF) modelling is a surface tracking technique adopted for multiphase flows. Multiphase flow generally consists of two immiscible fluids i.e. water and air, where the fluids are non-interpenetrating. The prime focus of VOF modelling is to track interface or free surface between the two fluids (for current study these are water and air). Therefore, two immiscible fluids can be modelled by solving a single set of momentum equations and volume fraction is tracked across the domain in each computational cell. New variable defined as volume fraction of the fluid, is introduced in each computational cell. The sum of all phases in each computational cell is unity.

Experimental conditions were used to specify computational domain for numerical simulation. The domain comprises of rectangular channel of 1.5m in length, 0.7 in width and 0.1m in height. The length of channel is reduced due to computational constraints. Since, the experimental channel of 5m requires more computational time and higher system requirements. Vegetation elements of 0.1m height and 0.004m diameter are modelled as rigid cylindrical elements. The cylinders are spaced according to vegetation density and aspect ratio.

The Ansys toolkit workbench was used to model cases. Multizone meshing was employed for discretization the flow domain into hexahedral cells. The mesh size was different for each case as it depends upon the vegetation conditions i.e. aspect ratio. Mesh independence was also performed to check independency of grids. The simulation setup is done in CFD code FLUENT which was then exported to CFD-POST for postprocessing. VOF implicit scheme was used for free surface tracking. Boundary conditions are shown in Fig. 3. The inlet of channel is divided into two phases that include water and air. Water phase at the start of channel was taken as velocity inlet while pressure inlet condition was applied to air phase. The free surface and outlet were modelled as pressure outlet and gauge pressure was set to zero. Other boundaries of channel domain that include side walls, bed, and cylinders were considered as no-slip wall condition. All vegetation models converged when the residuals were set to 1/1000.

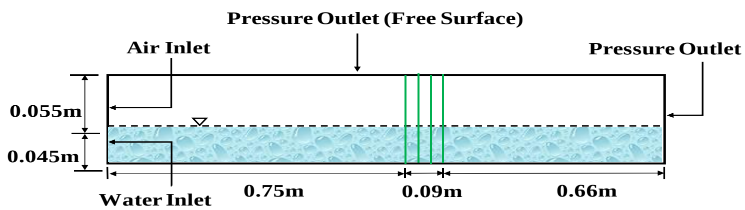


Fig. 3. Layout of flow domain and boundary conditions

2.4. Governing equations and turbulence model

Reynolds-Averaged Naiver-Stokes (RANS) equations are used to characterize flow behaviour in vegetated open channel. These equations are based on the principles of physics that include conservation of mass (basis for continuity equation) and Newton’s second law of motion (basis for momentum equation). k-ε is two equation turbulence model comprising of turbulence kinetic energy (k) and turbulence dissipation rate (ε) which was employed for modelling turbulence parameters of flow (Versteeg and Malalasekera, 1995). RANS equations for continuity and momentum are given as:

Continuity Equation

$$\frac{\partial \overline{u}_i}{\partial x_i} = 0 \tag{1}$$

Momentum Equation

$$\frac{\partial \overline{u}_i}{\partial t} + \frac{\partial}{\partial x_j}\left(\overline{u_j}\,\overline{u_i}\right) = -\frac{\partial \overline{p}}{\partial x_i} + \nu\frac{\partial^2 \overline{u_i}}{\partial {x_i}^2} - \frac{\partial}{\partial x_j}\left(\overline{{u_i}'{u_j}'}\right) \tag{2}$$

$\overline{u}i$ and $\overline{u}$j represents the averaged velocities and, ${u_i}'$and ${u_j}'$ represents the fluctuating velocities in xi and xj direction respectively. Over bar is used to denote Reynolds averaged parameter. Here, *t* is time in seconds, *p* is kinematic pressure in *Pa* and *v* is fluid kinematic viscosity in m^2/s.

Turbulence kinetic energy (k) Equation

$$\frac{\partial(\rho k)}{\partial t} + \frac{\partial(\rho k u_i)}{\partial x_i} = \frac{\partial}{\partial x_i}\left[\frac{\mu_t}{\sigma_k}\frac{\partial k}{\partial x_j}\right] + 2\mu_t E_{ij}E_{ij} - \rho\varepsilon \tag{3}$$

Turbulence dissipation rate (ε) Equation

$$\frac{\partial(\rho\varepsilon)}{\partial t} + \frac{\partial(\rho\varepsilon u_i)}{\partial x_i} = \frac{\partial}{\partial x_j}\left[\frac{\mu_t}{\sigma_\varepsilon}\frac{\partial\varepsilon}{\partial x_j}\right] + C_{1\varepsilon}\frac{\varepsilon}{k}2\mu_t E_{ij}E_{ij} - C_{2\varepsilon}\rho\frac{\varepsilon^2}{k} \tag{4}$$

$$\mu_t = \rho C_\mu \frac{k^2}{\epsilon} \tag{5}$$

Where Cμ is a dimensionless constant, ρ is density of fluid in *kg/m3*, μt is eddy viscosity in *Pa.s* and *Eij* represents component of rate of deformation. The equations 4,5 and 6 consist of five constants Cμ , σk , σε , C1ε and C2ε. Where, the values of these constants are taken as C_μ= 0.09, σ_k=1.00, σ_ε =1.30, $C_{1\varepsilon}$ =1.44, $C_{2\varepsilon}$ =1.92. Diffusivities of *k* and ε are related by Prandtl numbers σk and σε to the eddy viscosity μt. With the purpose of evaluating pressure term of the exact *k*-equation, constants C1ε and C2ε are used for the right proportionality between the terms in the *k* and ε equations.

3. Validation of numerical model

3.1. Validation of velocity and water surface profile

Flow properties that include water surface profile and velocity distribution were validated by experimental data. The velocity was measured at three distinct locations (strips) as illustrated by Fig. 4. Because of fluctuations in flow velocity, the PIV measurements were recorded at 0.8*D*, where *D* is water depth. Moreover, symmetrical conditions prevailed at either side of vegetation centre line. Therefore, PIV was mounted on one side of centre line. The velocity variations on vegetation upstream was observed and hence it was measured at three strips located at 0-1 (strip-1),1-2 (strip-2) and 3-4cm (strip-3) from vegetation front. Fig. 5 shows the validation of numerical model with experimental data for *AR*=1.01. Vertical axis shows normalized longitudinal velocity profile in the cross-stream direction while the horizontal axis shows the distance from vegetation centreline. The simulation results presented good agreement with experimental data within permissible error.

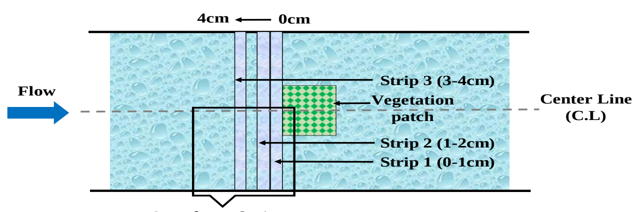


Fig. 4. Location of velocity measurement for numerical validation where, strip 1, strip 2 and strip 3 are 0-1cm, 1-2cm and 3-4cm respectively, from vegetation upstream.

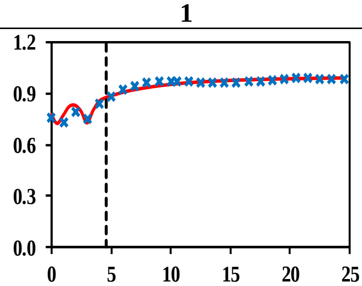
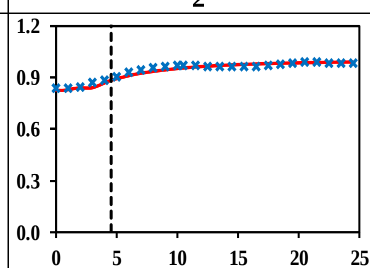
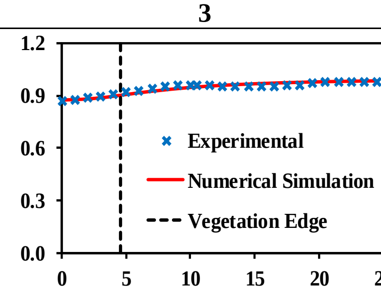
Case	***AR*=1.01**		
Strip	**1**	**2**	**3**
V_f/V_o			
	Distance from vegetation centreline (cm)	**Distance from vegetation centreline(cm)**	**Distance from vegetation centreline(cm)**

Fig. 5. Validation of longitudinal velocity distribution for strip 1, 2 and 3. Vertical axis represents normalized longitudinal velocity whereas horizontal axis represents distance from vegetation centreline.

4. Results and Discussions

4.1. Longitudinal velocity distribution

Fig. 6 shows the distribution of longitudinal velocity in cross-stream direction for all three vegetation cases including *AR* of 1.01, 1.70 and 2.40. These results represent the flow velocity directly in front of vegetation. The fluctuations in velocity on the upstream of vegetation are large due to turbulence. Moreover, the velocity recorded in front of vegetation is less as compared to gap between side wall and vegetation. The reason is that the water streamlines are reflected by vegetation cylinders and hence this resistance to flow causes decrease in velocity on vegetation front. However, the observed velocity in the gap is higher as the flow is unobstructed. Thuy et.al (2009) studied the effect of gap (void) between vegetation patches on flow behaviour. It was concluded that the gaps have negative effect on flow velocity. The variation in velocity for AR=1.01 extends to lesser distance as compared to *AR*=1.70 and *AR*=2.40. This is due to the length of vegetation which offers resistance to flow, is more in *AR*=2.40 than *AR*=1.70. Therefore, larger the aspect ratio more is the extent of vegetation resistance.

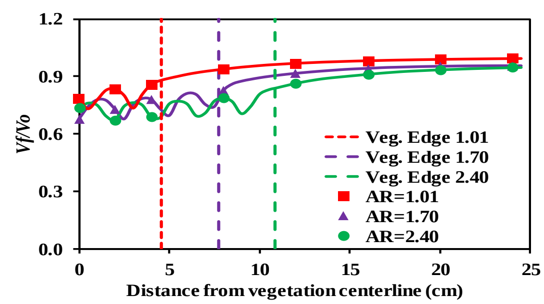


Fig. 6. Plots of longitudinal velocity distribution for *AR*=1.01, 1.70 and 2.40. Dashed lines represent vegetation edges for respective aspect ratio.

Fig. 7 shows the contours of velocity at critical locations i.e. vegetation upstream and downstream. It is evident from the figure that velocity upstream of vegetation is reduced due to flow hindrance by vegetation cylinders. However, the velocity inside the gaps are higher. The downstream velocity contours show that the velocity is reasonably reduced not only behind vegetation but also the effect of vegetation obstruction is observed inside the gaps. In addition to it, the extent of vegetation obstruction is greatest in *AR*=2.40 and least in *AR*=1.01. The water has to move faraway in large aspect ratio to reach edge of vegetation where it can gain velocity. Therefore, the effect of aspect ratio on flow velocity is significant.

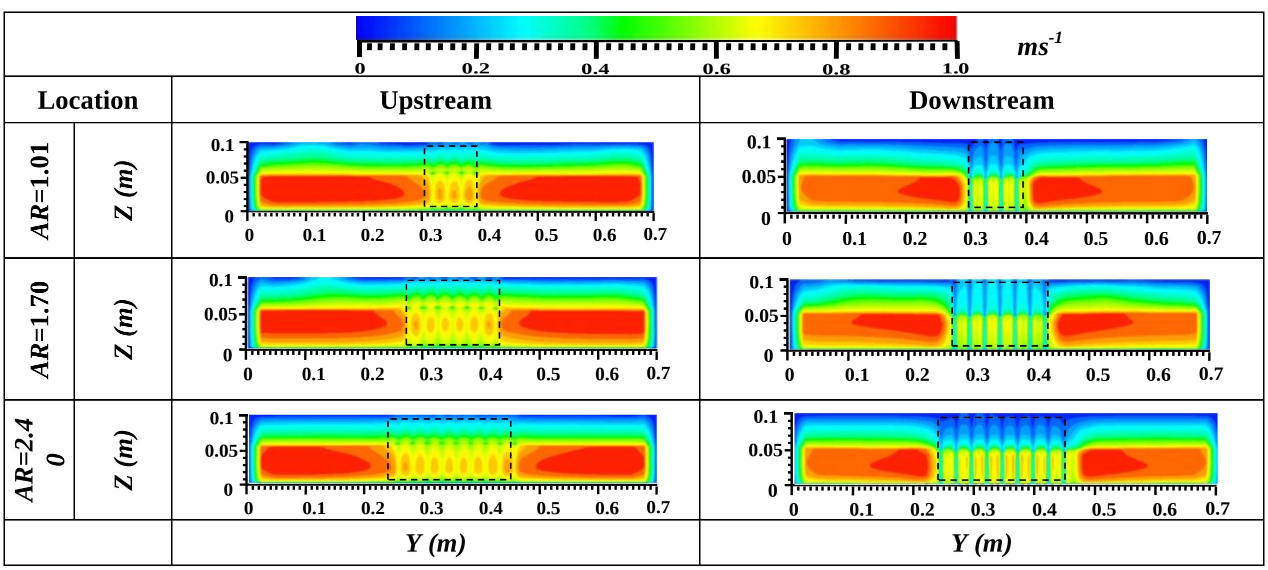


Fig. 7. Contours of longitudinal velocity distribution for *AR*=1.01, 1.70 and 2.40. *Y* represents channel width while *Z* represents height of channel. Dashed boxes show vegetation area.

4.2. Water Surface Profile

Fig. 8 shows the variation in water levels along channel width. Water levels are measured directly in front (upstream) of vegetation. Pasha and Tanaka (2019) employed an analytical approach to determine the water depths upstream, inside, and downstream of vegetation. The results show that the water level in front of vegetation is raised. This is due to the reason the reflection of water by vegetation elements which causes water to rise on the upstream of vegetation. However, the water levels in the gap are comparatively low. The aspect ratio influences not only the rise in water on vegetation upstream but also the extent to which this change occurs. Therefore, back water rise (rise in water level on vegetation upstream) is greatest in case of *AR*=2.40 and least in *AR*=1.01 whereas, *AR*=1.70 shows intermediate trend. Moreover, the difference in water levels in front of vegetation and gap is also highest in *AR*=2.40. Hence, the vegetation aspect ratio plays key role in obstructing flow.

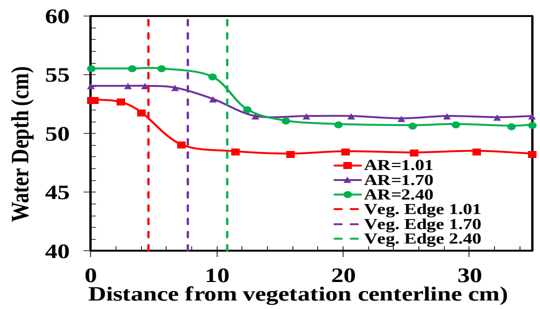


Fig. 8. Profile of water surface in cross-stream direction for *AR*=1.01, 1.70 and 2.40. Dashed lines represent vegetation edges for respective aspect ratios.

Fig. 9 shows the contours of water levels along channel length. Analysing water levels at critical locations indicate that the vegetation elements greatly affects water levels. As the flow approaches vegetation patch, the water streamlines face obstruction which results in backwater rise and consequently, the water level at vegetation downstream is reduced. This happen because the flow while moving through vegetation observes hindrance at each cylinder and as it reaches the end of patch, subsequently the water level is dropped. It is obvious from contours that the backwater-rise and downstream water level drop is greatest in case of *AR*=2.40 and least in *AR*=1.70.

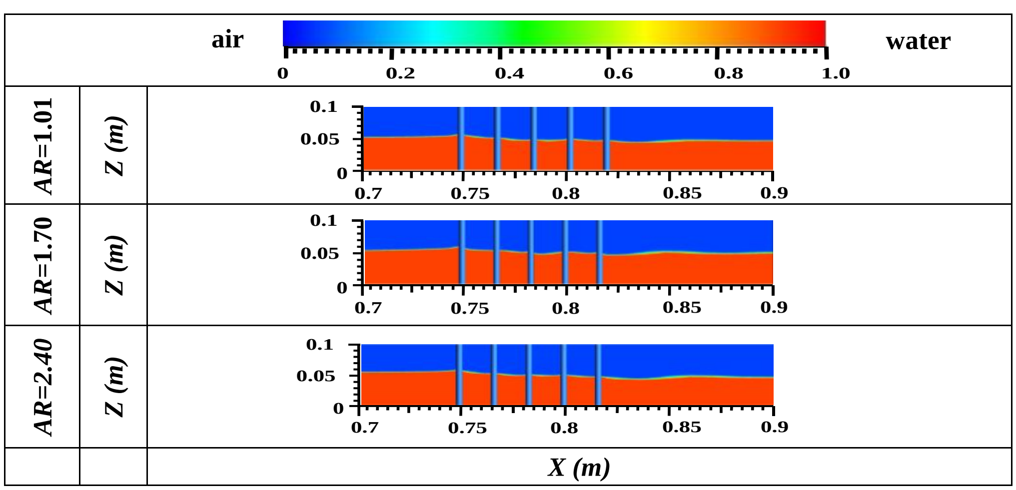


Fig. 9. Contours of longitudinal water surface profile for *AR*=1.01, 1.70 and 2.40. *Z* represents channel height while *X* represents distance from channel start.

5. Conclusions

The present study employed numerical investigation of flow through vegetation patches of varying aspect ratios. The simulation results were achieved by VOF multiphase modelling with standard k-ε turbulence model. The following conclusions were drawn from this research:

1. The CFD code FLUENT with multizone meshing presented good correspondence with experimental data which proves the applicability of software in solving engineering problems.
2. The variation in velocity distribution with three different aspect ratios (*AR*) i.e. 1.01, 1.7 and 2.40 was analysed. It was found that larger aspect ratio resists more flow and therefore, has greater effect on flood flow velocity as compared to smaller aspect ratio.
3. The velocity inside the gaps was observed higher. The difference in velocity between vegetation front and gap was highest in case of *AR*=2.40 and least in *AR*=1.01.
4. The water surface was raised in front of vegetation due to vegetation obstruction. However, the water level behind vegetation was dropped. Moreover, the effect of aspect ratio in backwater rise and its extent was significant in larger aspect ratio (*AR*=2.40) than smaller aspect ratio (*AR*=1.01).

Acknowledgments
The authors are grateful to Higher Education Commission (HEC), Pakistan for the provision of CFD software facility at University of Engineering and Technology Taxila, Pakistan.

References

Ahmed, A., & Ghumman, A. (2019). Experimental Investigation of Flood Energy Dissipation by Single and Hybrid Defense System. *Water, 11*(10), 1971.

Debnath, K., Bhattacharya, A., Mahato, B., & Chakrabarti, A. (2008). Volume of fluid model for numerical simulation of vegetated flows. *ISH Journal of Hydraulic Engineering, 14*(2), 72 87.

Igarashi, Y., & Tanaka, N. (2018). Effectiveness of a compound defense system of sea embankment and coastal forest against a tsunami. *Ocean Engineering, 151*, 246-256.

Iimura, K., & Tanaka, N. (2012, 11 1). Numerical simulation estimating effects of tree density distribution in coastal forest on tsunami mitigation. *Ocean Engineering, 54,* 223-232.

Pasha, G., & Tanaka, N. (2016). Effectiveness of Finite Length Inland Forest in Trapping Tsunami-Borne Wood Debris. *Journal of Earthquake and Tsunami, 10*(4), 1650008.

Pasha, G., & Tanaka, N. (2019). Critical Resistance Affecting Sub- to Super-Critical Transition Flow by Vegetation. *Journal of Earthquake and Tsunami, 13*(1), 1950004.

Ramamurthy, A., Qu, J., & Vo, D. (2007). Numerical and experimental study of dividing open-channel flows. *Journal of Hydraulic Engineering, 133*(10), 1135-1144.

Tanaka, N., Sasaki, Y., Mowjood, M., Jinadasa, K., & Homchuen, S. (2007). Coastal vegetation structures and their functions in tsunami protection: Experience of the recent Indian Ocean tsunami. *Landscape and Ecological Engineering, 3*(1), 33-45.

Tariq, M., & Van de Giesen, N. (2012). Floods and flood management in Pakistan. *Physics and Chemistry of the Earth, Parts A/B/C, 47*, 11-20.

Thuy, N., Tanimoto, K., Tanaka, N., Harada, K., & Iimura, K. (2009). Effect of open gap in coastal forest on tsunami run-up—investigations by experiment and numerical simulation. *Ocean Engineering, 36*(15-16), 1258-1269.

Västilä, K., Järvelä, J., & Aberle, J. (2013). Characteristic reference areas for estimating flow resistance of natural foliated vegetation. *Journal of hydrology, 492,* 49-60.

Versteeg, H., & Malalasekera, W. (1995). Computational fluid dynamics. *The finite volume method*.

Zaha, T., Tanaka, N., & Kimiwada, Y. (2019). Flume experiments on optimal arrangement of hybrid defense system comprising an embankment, moat, and emergent vegetation to mitigate inundating tsunami current. *Ocean Engineering, 173*, 45-57.

Zhan, J., Zhang, J., & Gong, Y. (2016, 5 1). Numerical investigation of air-entrainment in skimming flow over stepped spillways. *Theoretical and Applied Mechanics Letters, 6(3),* 139-142. Elsevier Ltd.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

To Investigate the Breaching Phenomenon of a Fuse plug Spillway

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Abstract

The safety of an earthen dam is of prime importance. Fuse plug spillway, a pre-defined breach section, works as a safety valve for an earthen dam. Flow over-topping crest of pilot channel can completely wash out the Fuse plug spillway. This Fuse plug spillway acts as a broad-crested weir after the erosion caused by the over-topping of flow. This research is unique in regard usage of local soil material i.e. Khanpur soil for investigating breaching phenomenon of Fuse plug spillway. Laboratory experiments were conducted in Water Resources Engineering Laboratory of Civil Engineering Department, University of Engineering and Technology Taxila, with varying designs of Fuse plug spillway. The first design of Fuse plug spillway included the inclined clay core within the embankment, while in the second design of the Fuse plug spillway; only seepage protection was provided without the inclined core. Fuse plug spillways eroded rapidly once they were over-topped. The Fuse plug spillway with inclined core showed that the breaching occurred in a progressive manner as a result of undercutting and collapse of the impermeable core, whereas the Fuse plug spillway without inclined core nearly breached continuously as there was no provision of inclined core. The Fuse plug spillway design without inclined core is recommended as the most appropriate for installation in the field because of its simple structure and ease in construction.

Keywords: *Earthen dam, Fuse plug spillway, flow overtopping, breaching, undercutting, impermeable*

1. Introduction

With rapid population growth infrastructure demand has increased significantly. Infrastructure development has also been linked to the economic growth of a country. Dams are an essential part of infrastructure. Being a mega infrastructure dam's safety is of prime importance. This purpose is being fulfilled by the provision of Fuse plug spillway.

A Fuse plug spillway is a zoned earth and rock filled embankment designed to wash out in a predictable and controlled manner when the flow capacity needed exceeds the normal capacity of the service spillway and the outlet works. A Fuse plug spillway embankment is designed to prevent use of the auxiliary spillway during minor floods, much the same as spillway gates. In many cases, auxiliary spillways with Fuse plug spillway embankments can provide an economical alternative to passage of all the flow through concrete structures.

Figure 11: Fuse plug (left side in the photo) at Warragamba dam in NSW Australia (www.waternsw.com)(Lagerlund. Johan (2018))

The discharge capacity of Fuse plug spillway is directly proportional to the rate of erosion of the Fuse plug spillway. In this way the main body of the dam will be shielded from overtopping and would be safe against failure. The boundary line between conventional embankment and Fuse plug spillway is the material composition of downstream shoulder. More easily erodible material can be used for Fuse plug spillway. Several experimental researches were carried on the breaching of an earthen embankment. Amongst others Lagerlund (2018), Pagh (1985), Schmocker et al (2013), Babaali and Shamsai Abolfazl (2012) studied the breaching of Fuse plug spillway. Flow over topping causing breaching studied by Hanson.G.J et al. (2005*)*, Amaral et al.(2014), Chongxun Mo et al.(2008), Hiller P.H et al. (2018), Tabrizi et al. (2016), Yun Zhang Jian et al. (2009) and Zhong et al. (2018).
The aim of this experimental research is to investigate the breaching phenomenon of Fuse plug spillway. For this purpose two different Fuse plug spillways were designed as zoned earthen embankments. One with inclined core and the other one without inclined core. The breaching advancement with time was investigated for both of them. The readings were compared to select the best out of them.

2. Experimental Program

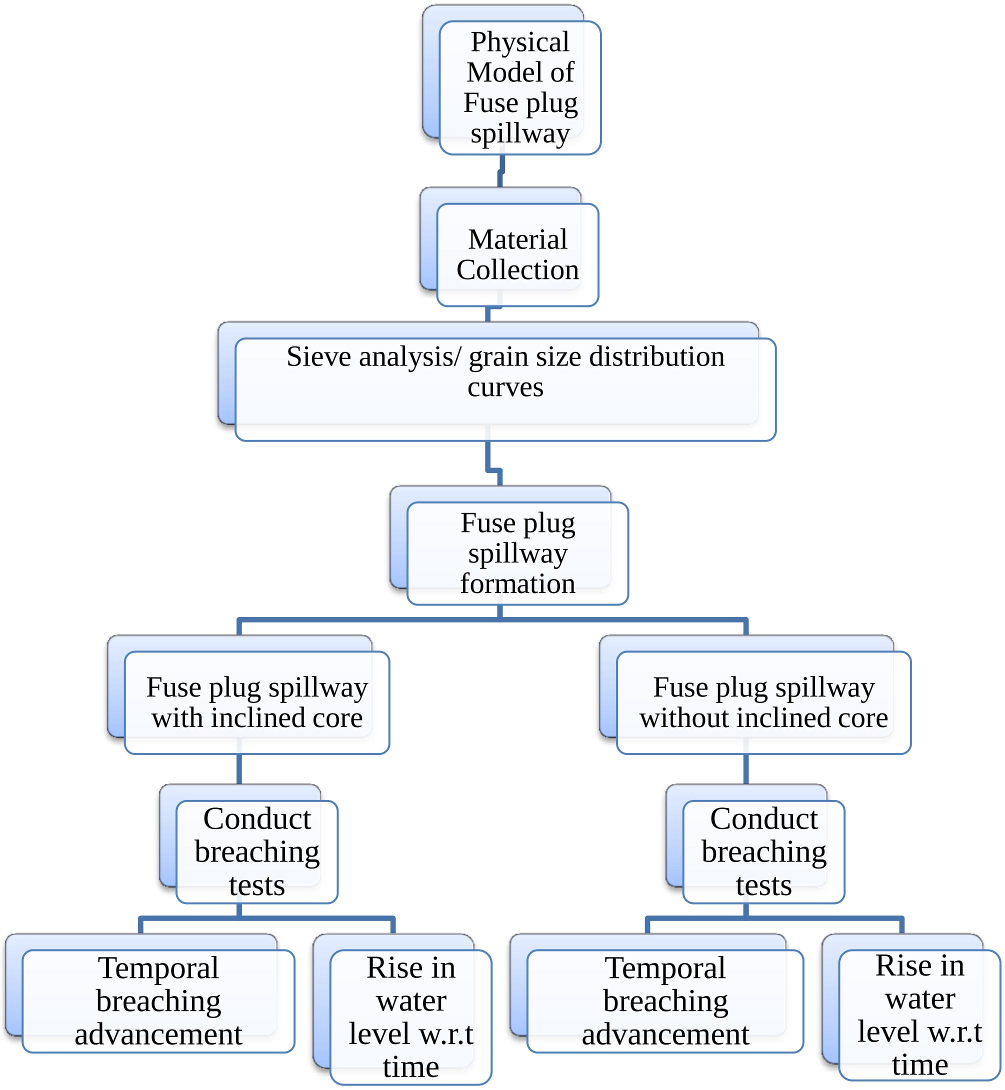


Figure 12: Methodology and approach of this study

2.1 Flume description

Tests were performed in the Water Resources Engineering Laboratory of Department of Civil Engineering, University of Engineering and Technology, Taxila. The hydraulic flume used for experimental was 20 m long, 0.96 m wide and 0.75 m deep, provided with glass side walls and concrete bottom. The discharge is supplied through pump from a side tank by aligned honeycomb diffuser in the direction of flow for smooth and uniformly distribution of flow cross-wise. The experiments were carried out 6 meter downstream of the discharge diffuser. The discharge was kept constant at 0.02 m^3/sec.

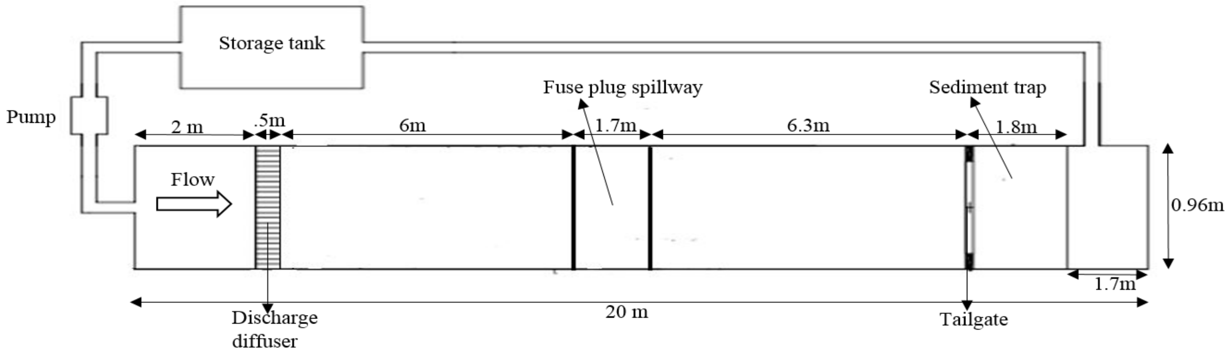


Figure 13: Plan View of Hydraulic Flume

2.2 Design of Fuse plug spillway

Fuse plug spillway spillways were designed and constructed as a zoned embankment. Pilot channel was given in the center of crest for the initiation of breaching. The width of pilot channel was set as 0.5*H* (Pugh 1985), where *H* is the total height of Fuse plug spillway and thickness was set as 5cm.

2.2.1 Design of Fuse plug spillway without inclined core

For the first set of calculations the Fuse plug spillway was designed with clay as a seepage protection, sand filter, fine gravel as a body and coarse gravel as a slope protection. The schematic diagram Figure 4(a) was drawn in accordance to the literature given by Schmocker et al (2013) and side view of Fuse plug spillway without inclined core in Figure 4(b).

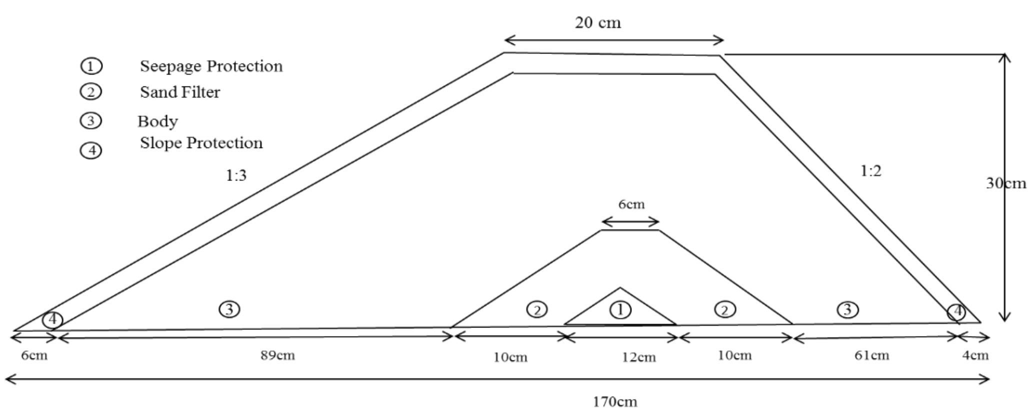


Figure 4 (a): Schematic diagram of cross-section of Fuse plug spillway without inclined core

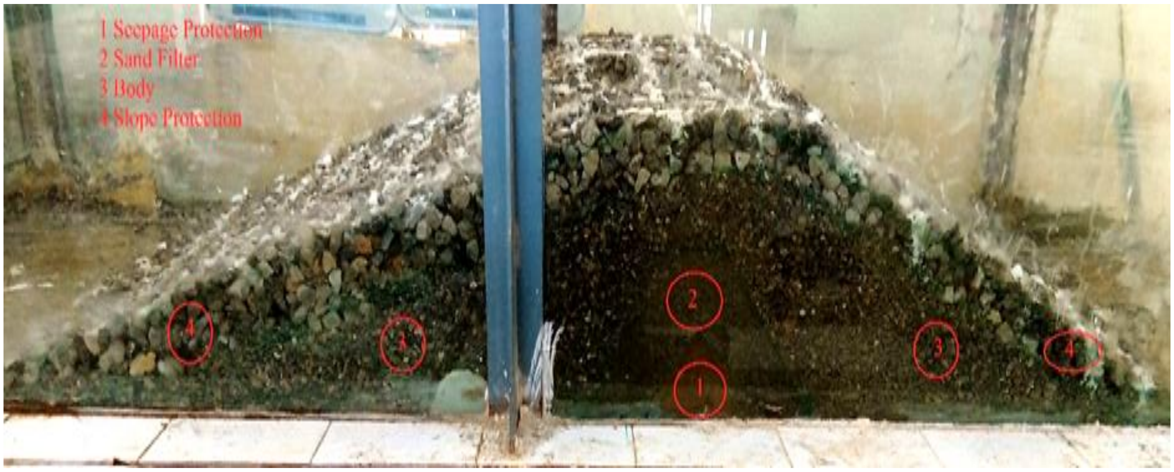


Figure 4 (b): Side view of Fuse plug spillway without inclined core

2.2.2 Design of Fuse plug spillway with inclined core

For the second set of calculations the Fuse plug spillway designed and constructed provided 45˚ inclined clay core, instead of seepage protection. The schematic diagram and side view of Fuse plug spillway with inclined core are shown in Figure 5(a) & (b).

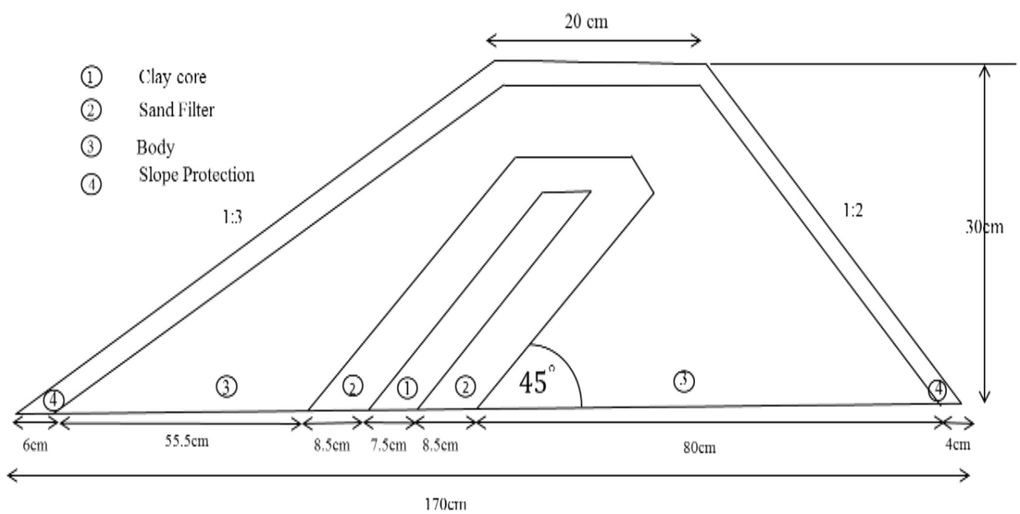


Figure 5(a): Schematic diagram of Cross-section of Fuse plug spillway with inclined core

Figure 5(b): Side view of Fuse plug spillway with inclined core

2.2 Material Properties

To know the grain size of the material being used for the different parts of the embankment Sieve analysis was carried out. The Grain size distribution curves in the Figure 6.

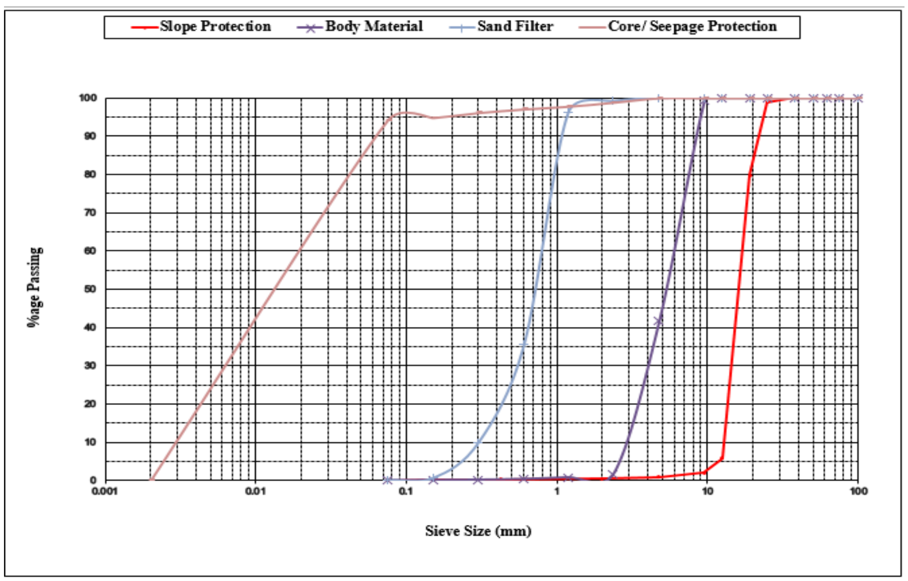


Figure 6: Grain size distribution curves

From the above sieve analysis and gradation curve it was observed that different materials have different grain sizes. The core and seepage protection material was highly plastic clay (CH), Filter material was poorly graded sand (SP), well graded gravel (GW) for embankment body material and poorly graded gravel (GP) for slope protection.

Material properties of Fuse plug spillway are given in Table 1:

Table 3 Material Properties of the Fuse plug spillway

Material	Hydraulic Conductivity, K_{sat} (m/s)	Unit weight above phreatic line, γ_u (kN/m^3)	Effective Friction angle Φ (˚)	Effective Cohesion c (kPa)
Core/seepage Protection	$1.0x10^{-2}$	21	38	20
Filter	$1.0x10^{-2}$	21	32	0
Body Material	$1.0x10^{-2}$	21	34	0
Slope protection	$1.0x10^{-2}$	19	30	7

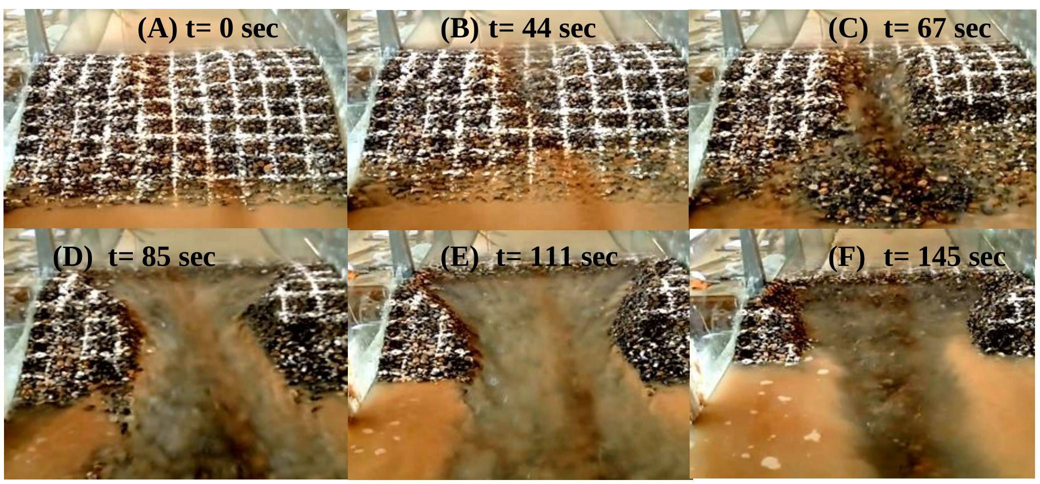
The material properties in Table 1 were derived based on grain size distribution. Hydraulic conductivity, unit weight above phreatic line, effective friction angle and effective cohesion for core, filter, and body material and slope protection were obtained from literature based on similar types of soils (Vahdati 2014).

3. Results and Discussions

3.1 Fuse plug spillway without inclined core

The experimental calculations showed that the seepage occurred prior to the flow overtopping. The breaching phenomenon is described in pictorial form given in Table 2.

Table 2: Temporal breaching advancement of Fuse plug spillway without inclined core

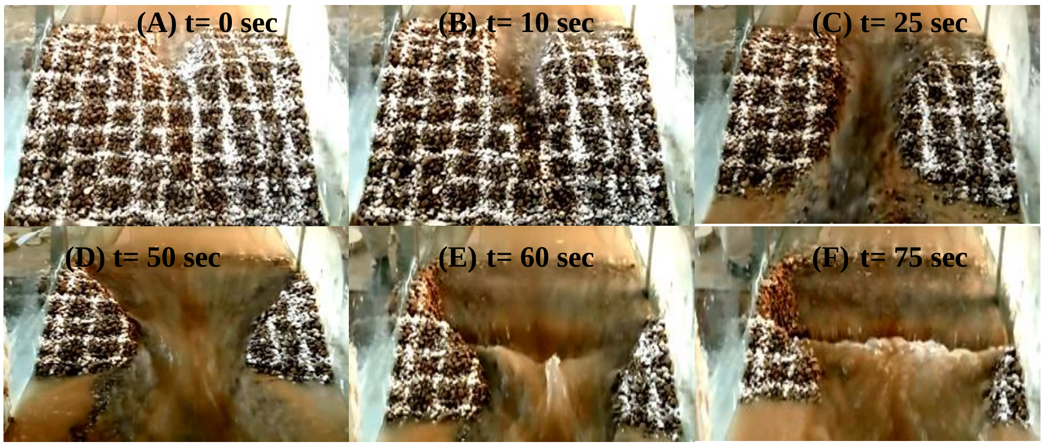


The breaching was initiated at t= 44seconds and test aborted at t=145 seconds and almost 70% area of downstream side was washed away. The reading showed that breaching was continuous and gradual.

3.2 Fuse plug spillway with inclined core

Experimental calculation showed that there was negligible seepage downstream at the start. The breaching of the embankment is represented in pictorial form in Table 3.

Table 3: Temporal breaching advancement of Fuse plug spillway with inclined core



The breach was initiated at t= 10 second and test aborted at t= 75seconds and almost 60% area downstream side washed away. These readings showed that the breach was progressive in nature.

3.3 Comparision of Fuse plug spillways with and without Inclined Core

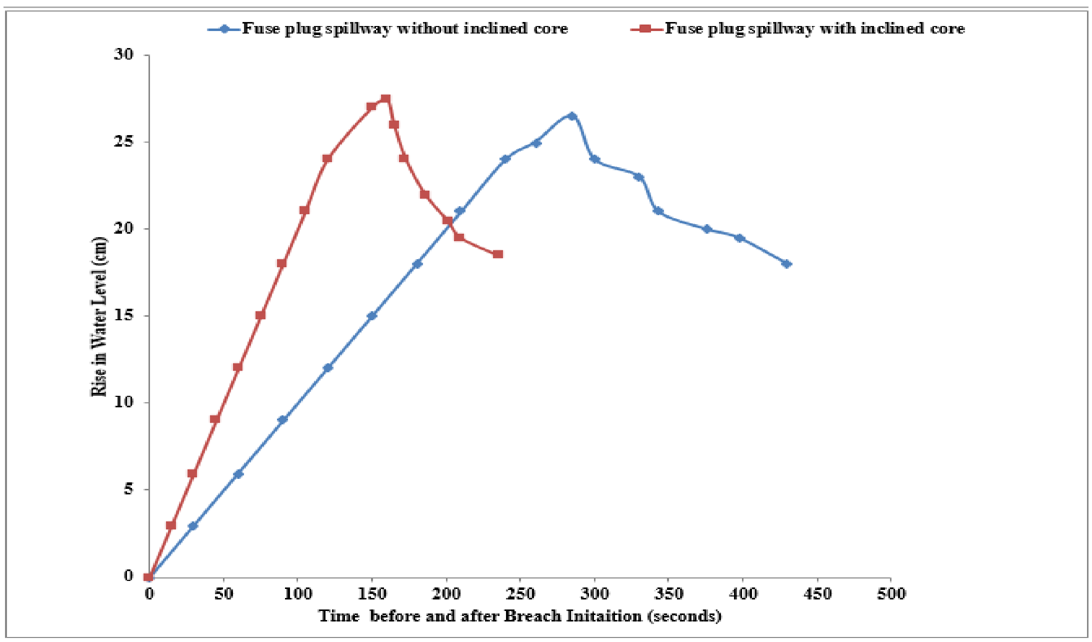


Figure 6: Rise in water level of reservoir before and after Breach Initiation with respect to time

The rise in water level was rapid and high in case of Fuse plug spillway with inclined core which was 27.5cm while in Fuse plug spillway without inclined core it was noted to be 26.5cm. But the Fuse plug spillway with inclined core breached in less time as compared to the Fuse plug spillway without inclined core.

4. Conclusion

In this experimental study, the breaching phenomenon of Fuse plug spillway was investigated. The experiments on Fuse plug spillway with and without inclined core were carried out.

- At the start seepage was observed in the Fuse plug spillway without inclined core prior to the breach initiation but when the breach was initiated the seepage as well as the breaching was continuous and gradual.
- While in the other case of Fuse plug spillway with inclined core, negligible seepage was noticed prior to the breach initiation; however, once started, it was progressive.
- The conclusion drawn from the results is that the Fuse plug spillway without inclined core, with proper seepage protection, is preferred because of its simple structure and ease of construction.

5. References

Amaral et al.(2014) "Failure by overtopping of earth dams. Quantification of the discharge hydrograph." *3rd IAHR Europe Congress, Book of Proceedings*, 2014, Porto – Portugal.

Babaali Hamid Raza, Shamsai Abolfazl (2012) **"**The Comparison of the Design of Fuse plug Spillways from the Aspect of Erosion and Stability." *Journal of Basic and Applied Scientific Research* 2012.

Chongxun Mo et al.(2008) "Risk analysis for earth dam overtopping." *Water Science and Engineering,* Jun. 2008, Vol. 1, No. 2, 76–87

Hanson.G.J et al. (2005*) "*Physical Modeling of overtopping erosion and Breach formation of cohesive embankments.*" American Society of Agricultural Engineer/*Vol.48/No.5/July, 2005(1783-1794)

Hiller P.H et al. (2018) **"**Field and model tests of riprap on steep slopes exposed to overtopping". *Journal of Applied Water Engineering & Research*, 2018

Lagerlund. Johan (2018) "Fuse plug spillway Breach Tests: Can a laboratory scale test replace a field test?" Report 2018: 465

Pagh.Clifford A (1985) "Hydraulic Model Studies of Fuse plug spillway embankments" REC-ERC-85-7

Schmocker et al (2013) "Hydraulic Model Study of the Fuse plug spillway at Hagneck Canal, Switzerland*" Journal of Hydraulic Engineering*,Vol. 139, No. 8, August 1, 2013. © ASCE, ISSN 0733-9429/2013/8-894-904/$25.00

Tabrizi et al. (2016) "Effects of compaction on embankment breach due to overtopping." *Journal of Hydraulic Research, 2016.*

Vahdati, P. (2014) "Identification of soil parameters in an embankment dam by Mathematical optimization." Licentiate thesis. ISSN 1402-1757. Luleå: Luleå University of Technology.

Yun Zhang Jian et al. (2009) "Overtopping breaching of cohesive homogeneous earth dam with different cohesive strength." Sci China Ser E-Tech Sci, 2009, 52(10): 3024**—**3029

Zhong et al. (2018) "A simplified physically-based model for core dam overtopping breaches." *Engineering Failure Analysis* 90 (2018) 141–155

11th International Civil Engineering Conference (ICEC-2020)
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March 13-14, 2020, Karachi, Pakistan.

Effects of Stirrup's spacing on the confinement, ductility, strength and curvature of circular column using Matlab cumbia code

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Abstract

This study was carried out in Khyber Pakhtunkhwa, District Peshawar after strongly felt earthquake of magnitude 5.6 in most parts of the Upper Punjab (Mangla, Dina, Lahore, Kharian, Gujjar Khan, Gujrat, Hafizabad, Lala-Musa,) and AJK (Mirpur, Muzaffarabad). Most of the structure fails due to fragile behavior and profound confinements. Almost 40 people losses their lives and 850 injured. This research work fully focuses on the importance of the enhancement of the designed parameters of the structures. The different sample of each prototype columns with stirrup's spacing 3″, 6″, 9″ and 12″ are processed through Matlab Cumbia code. The results of the Matlab Cumbia code for each stirrups spacing on the confinement, ductility, strength and curvature of the axially loaded confined reinforced concrete prototype columns are presented. The comparative study of different stirrups spacing on the concrete column capacity, ductility and curvature have been researched.

Keywords

Concrete, Confinements, Ductility, Experimental testing of Columns, Stirrups, Strength.

1. INTRODUCTION

The enhancement in the confinement of the column concrete is noteworthy in preparing seismic resilient design of the structures in two ways. Firstly the column concrete will be designed for large deformations so that it may able to withstand larger earthquakes. Secondly, when the spalling of the concrete cover starts the strength and ductility of columns furthermore depends on confinement of column concrete core.
The most suitable and sufficient confinement of the core of the column with the transverse reinforcement helps us in the achievement of the requisite bearing capacity and plastic rotation, till high angle of curvature. For ensuring of the appropriate resistance to the earthquake i.e. seismic forces, it is particularly important to provide a significant quantity of transverse reinforcement bars in that area of column anticipated with plastic hinges. Because it will enhance the ductility factor and load carrying capacities. In case of local bucking of longitudinal reinforcement of long and intermediate columns, the fractures in concrete are due to transverse forces. At the time of complex bending of columns due to normal compressive forces, the ductile properties and characteristics are achieved through nonlinear deformations

of the compressed part of concrete, the compressed portion of concrete cross-section increases. Confined and unconfined zone of concrete is directly proportional to the number of stirrups. When the density of the stirrups is increased in a unit length of the column, confinement of the column increases along with strength.

2. DESCRIPTIONS AND REQUIRED INPUTS FOR TESTED COLUMNS

The clarification and input data that are required for the pre-processing of the program are divided into the following categories. During the initial stage section properties i.e. diameter of the section, clear cover of the longitudinal bars is to be acknowledged. Selection properties will be followed by material properties. In this phase we have to provide the length of the column, bending and ductility mode as a uniaxial or biaxial. The reinforcement details are too supplied after member properties. The number and diameter of longitudinal reinforcement bars, the diameter, type and spacing of the transverse reinforcement bar is provided in this phase. After completion of the preceding step the applied load will be given. In the succeeding steps the materials properties as well as the deformation limit states will be provided. After completion of the pre-processing run the program, post processing will start. The inputs parameters are shown in table 1.

Table 4: Input Parameters for Matlab Cumbia code

Case	Stirrups	f_y (Ksi)	f_c (Ksi)	Column 1	Column 2
1	#3@ 3″ c/c	60	3	8 # 6	8 # 6
2	#3@ 3″ c/c	60	2	8 # 6	8 # 6
3	#3@ 6″ c/c	60	2	8 # 6	8 # 6
4	#3@ 9″ c/c	60	2	8 # 6	8 # 6

3. RESULTS AND DISCUSSIONS

The results of two columns obtained from Matlab Cumbia code under different axial loads, confinements and compressive strength are shown in table 2. Case 1 represent the compressive strength of the concrete as 3 ksi, whereas in remaining cases the compressive strength are kept as 2 ksi. The yield strength of the reinforcement bar is on average 60 ksi. Both of the columns contain equal longitudinal reinforcement bars, with different transverse reinforcements. The estimated values in the table 2 also represents yield and ultimate curvature, ductility, yield moment and ultimate moment of the different samples.

Table 5: Represent results of columns having different compressive strength and confinements.

MATLAB CUMBIA CODE RESULTS	**Case 1**		**Case 2**		**Case 3**		**Case 4**	
	Column 1	**Column 2**	**Column 1**	**Column 2**	**Column 1**	**Column 2**	**Column 1**	**Column 2**
Yield Curvature (1/m)	0.0182	0.01725	0.0196	0.01885	0.0197	0.01882	0.01692	0.01585
Ultimate Curvature (1/m)	0.4941	0.4409	0.5004	0.4911	0.1707	0.1877	0.4513	0.4144
Ductility	27.15	25.56	25.53	26.05	8.66	9.97	26.67	26.15
Yield Moment (KN-m)	116.4	107.9	106.9	102	106.3	101.8	92.68	84.64
Ultimate Moment (KN-m)	123.1	119.7	119.2	116.9	124.4	127.3	103.1	100.1

A). PROPORTION OF CONFINED AND UNCONFINED CONCRETE

The stress and strain relation for confined and unconfined concrete can be determined in the very first stage. During this stage the stress and strain curves of all the samples is estimated and compare for different stirrups spacing. The stress and the strain confinement ratio is also calcuted from the below bar graphs as represented in figure 2.

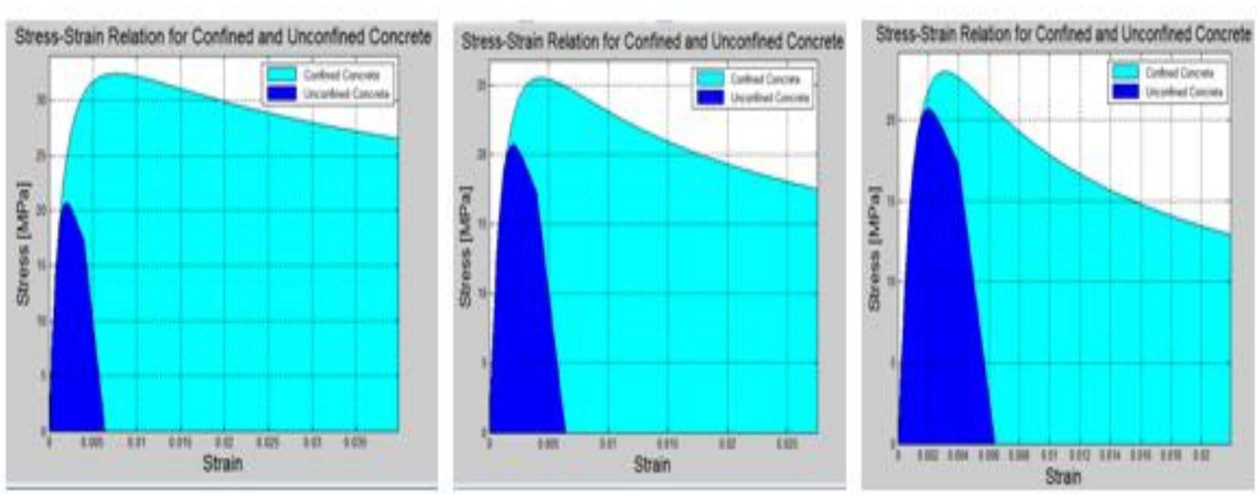


Figure 14: Columns with center to center stirrups spacing 3", 6" and 9"

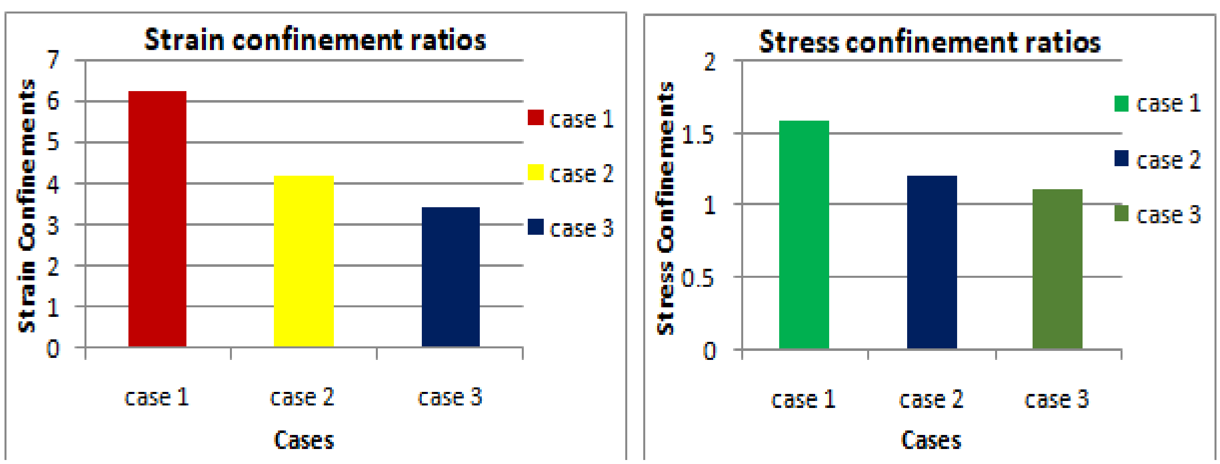


Figure 15: Strain and Stress Confinement Ratios of different cases

B). INFLUENCE OF THE CONFINEMENT ON THE DUCTILITY OF COLUMNS

The ductility of the reinforced concrete columns can be increased by providing appropriate confinement. This enhancement can also be accomplished by increasing the number of ties, increasing the strength of the ties, increasing yield strength of ties, proper arrangement of the longitudinal bars. This research work focuses on three fundamental methods. During the first two cases the proposed strength of the column concrete reduces from 3000 psi to 2000 psi keeping stirrups spacing constant. During the third case strength of the concrete kept constant and increase in spacing is carried out. During the fourth case, both the space and yield strength of the stirrups increased. The results for these cases are shown below.

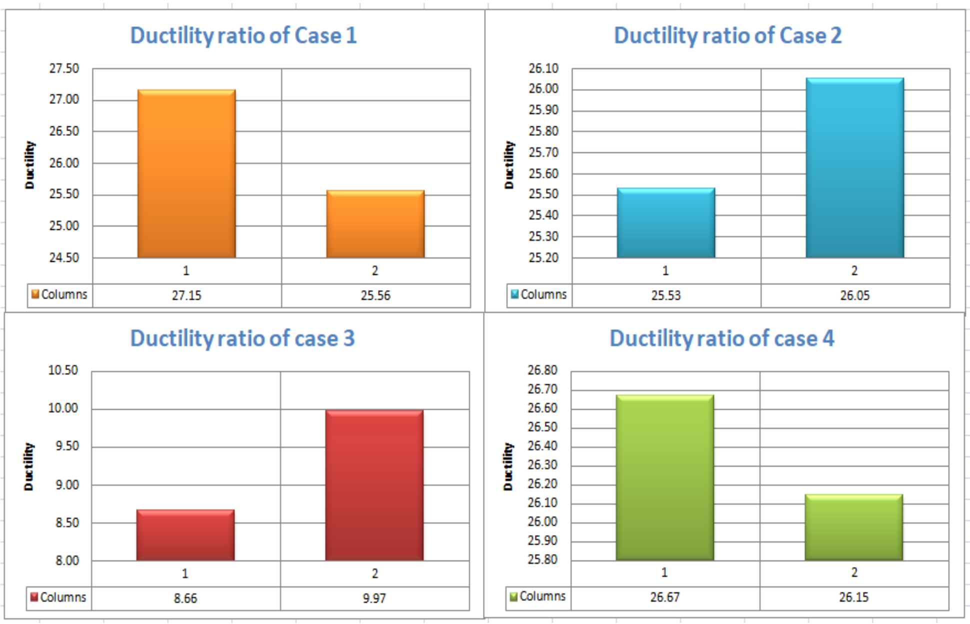


Figure 16: Ductility ratio's of columns containing different compressive strength and confinements.

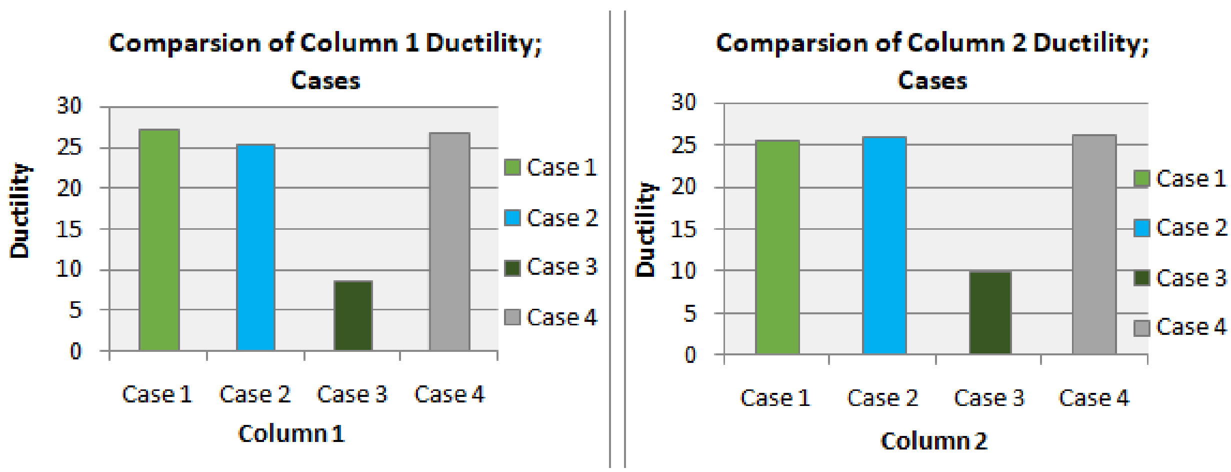
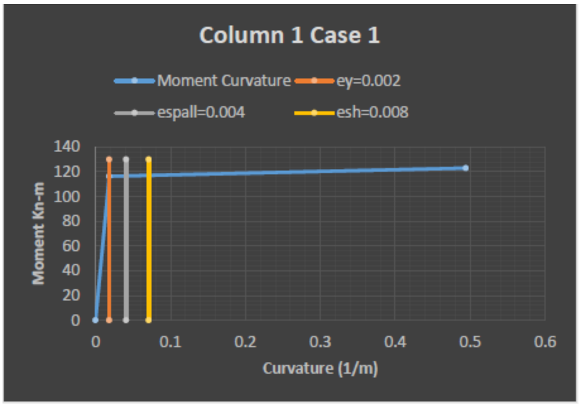


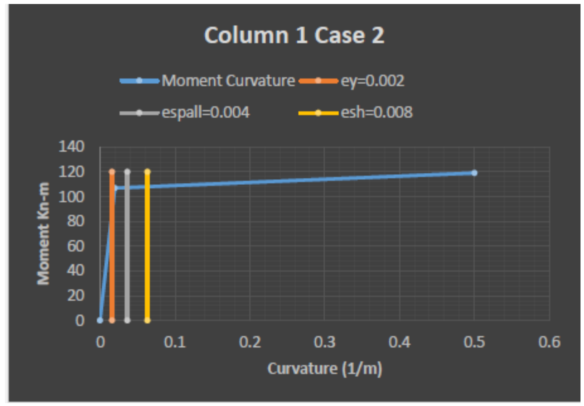
Figure 17: represents ductility of combine cases having different compressive strenth and confinement ratios

C). MOMENT CURVATURES OF COLUMNS

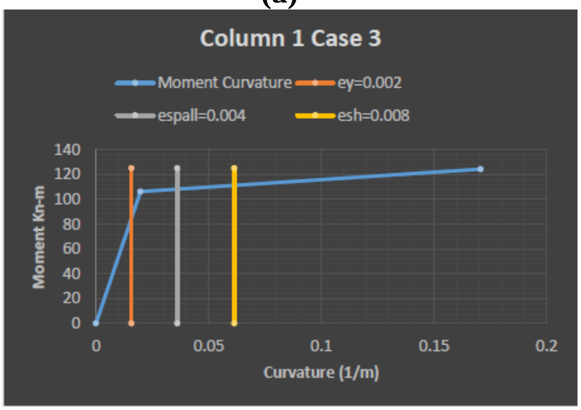
The performance of the structure under different loads largely depends upon the Moment-Curvature relationship and the materials comprising the system from which it is made. It also depends upon the stresses generated in the materials. In mostly cases concrete is used in compression so the primary interest is stress-strain and moment curvature curves. The moment curvature is of primary interest as shown in figures for every case.


Column 1 Case 1
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

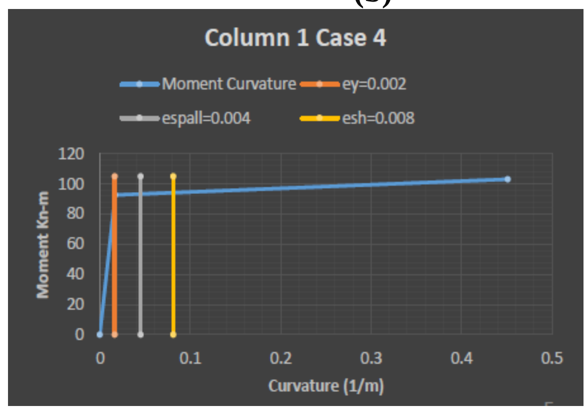
(a)


Column 1 Case 2
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

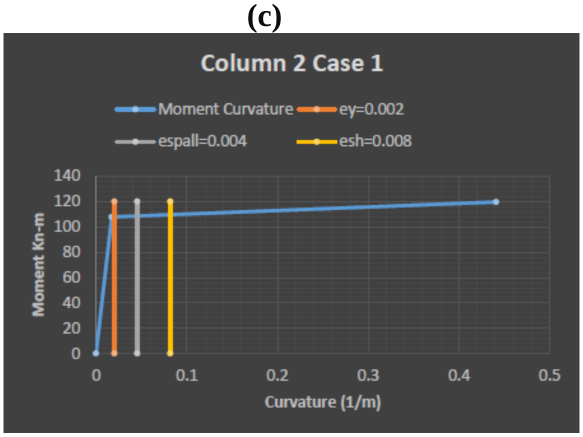
(b)


Column 1 Case 3
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

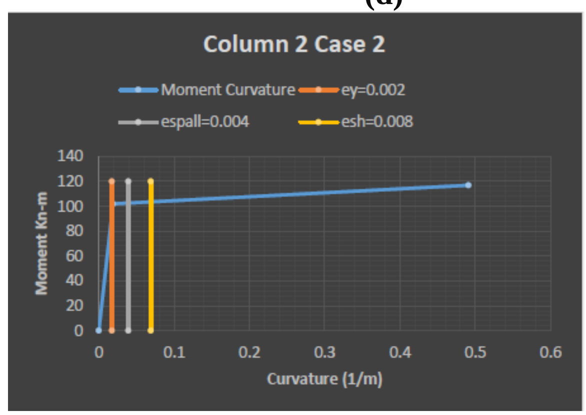
(c)


Column 1 Case 4
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

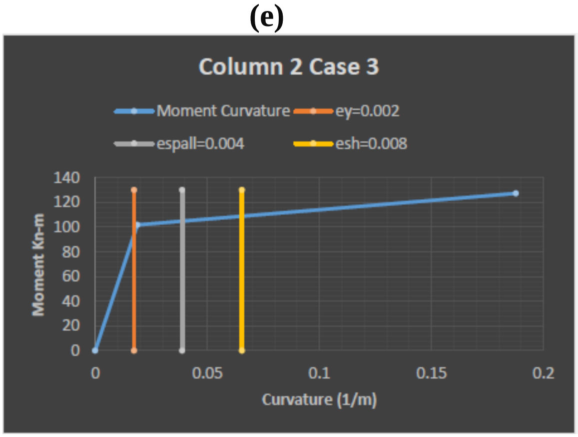
(d)


Column 2 Case 1
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

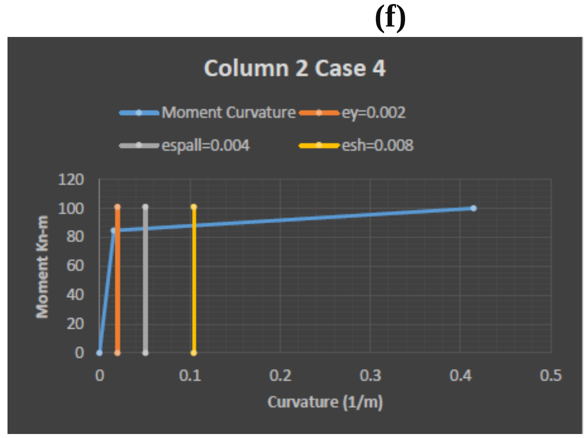
(e)


Column 2 Case 2
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

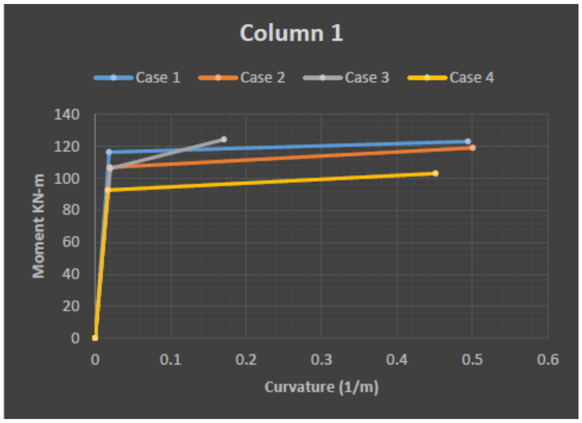
(f)


Column 2 Case 3
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

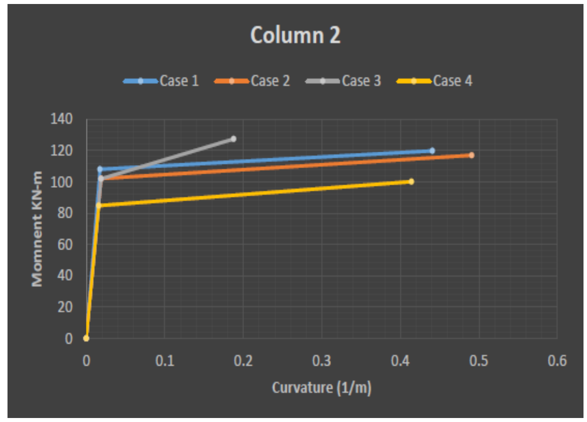
(g)


Column 2 Case 4
Moment Curvature
ey=0.002
espall=0.004
esh=0.008
Moment Kn-m
Curvature (1/m)

(h)



(i)



(j)

Figure 18: Represents the moment curvature relationships of different cases named from (a) to (j)

5. CONCLUSIONS

Based on the above investigations the following appropriate results are obtained during the testing.

- Both Yield and Ultimate Curvature of the concrete column increases with the increase in the compressive strength of the concrete.
- Ductility ratio of the column increases with the decrease in the center to center spacing of the stirrups provided in the column.
- Yield and Ultimate moment increases with the decrease in the center to center spacing of the stirrups provided in the column.
- Ductility ratio also increases with the increase in the yielding strength of the stirrups.
- Yield and Ultimate moment increases with the increases in the cross-section of the column.
- Spalling of the concrete take place at a curvature of 0.004
- Strength of the column increases with decreases in confinement of the columns.

6. REFERENCES

Paultre P, Legeron F, Mongeau D (2001) Influence of concrete strength and transverse reinforcement yield strength on behavior of high-strength concrete columns. ACI Struct J 98(4): 490–501

Mander, J.B., Priestley, M.J.N., Park, R.: Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Engineering, Vol. 114, No. 8, pp. 1804-1826, August 1988.

Paultre P, Eid R, Robles HI, Bouaanani N (2009) Seismic performance of circular high-strength concrete columns. ACI Struct J 106(4):395–404

Saadatmanesh, H., Ehsani, M.R., Li, M.W.: Strength and Ductility of Concrete Columns Externally Reinforced with Fiber Composite Straps, ACI Structural Journal, V. 91, No. 4, pp. 434-447, July.-August. 1994.

Cusson D, Paultre P (1993) Confinement model for highstrength concrete tied columns, Report No. SMS 9302. Department of Civil Engineer, University of Sherbrooke, Sherbrooke

A. M. el Fattah: Behavior of Concrete Columns under various Confinement Effects, Abstract of a Dissertation, M.S., Kansas State University, 2012

Mendis PA, Kovacic D, Setunge S (2000) Basis for the design of lateral reinforcement for high-strength concrete columns. Struct Eng Mech 9(6):589–600

Paultre, P., Legeron, F.: Confinement Reinforcement Design for Reinforced Concrete Columns, Journal of Structireal Engineering, may 2008

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Comparative Study on Seismic Analysis and Design of High-Rise Buildings using Static and Dynamic Analysis by ETABS

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ABSTRACT
This research focuses on seismic behavior of high-rise buildings. Building under study is Fortune One Tower located at Jinnah Avenue Sector F9 Islamabad (Seismic zone 2B) having 22 Stories with 04 basements. The project aims at finding the economical sizes of structural members and provide safety, stability and serviceability in the proposed building. The building was modelled in ETABS 2016 (Extended Three-Dimensional Analysis of Building System Version 2016). Linear Static and Linear Dynamic Analysis were performed on the selected building. Equivalent Lateral Force method was used as static analysis while Response Spectrum Analysis was used as Dynamic Analysis. Building Code of Pakistan (BCP), Universal Building Code (UBC-97) and American Concrete Institute (ACI 318-14) codes were followed for the design of the building. The outcome of this research is the economical sizes of structural members. Results established the superiority of Response Spectrum Analysis over the Equivalent Lateral Force Method.

Keywords:
Static Analysis, Dynamic Analysis, Equivalent Lateral Force & Response Spectrum Analysis.

1. INTRODUCTION

Earthquake is a natural cataclysm responsible for millions of deaths around the world since ancient times. The trembling of the earth due to an underground movement of tectonic plates along a fault plane or from volcanic activity is what we called an earthquake. Pakistan is situated in a region of high seismicity, which is evident from the catastrophic seismic event of October 8, 2005(Naseer *et al.*, 2010). An earthquake of 7.6 Magnitude shook Kashmir and Northern parts of Pakistan resulting 73000 deaths, more than 70000 injured or disabled, 2.8 million people become homeless, 2.3 million people are left with inadequate food and a total loss of 5.2 billion dollars to the economy of Pakistan (Durrani *et al.*, 2005). The possibility of repetition of such devastating earthquakes even greater than Magnitude of 8 are expected in the region(Avouac *et al.*, 2006).
The population of world is increasing, so is the case with urbanization, as a result vertical construction is necessary to accommodate the increased population and to avert the diminishing agricultural lands

(Ibrahim, 2007) but earthquakes have greater potential to damage high-rise buildings. Since forces developed under earthquakes are unpredictable and random in nature, the engineering tools need to be sharpened for analyzing such structures. Loads acting on a building may be static like load of people, furniture, snow, dead load of the building itself etc., or dynamic like earthquake forces, wind etc. Dynamic forces can be distinguish from static forces by their ability to produce acceleration with respect to the natural frequency of the structure (Kakpure and Mundhada, 2017). Designers usually perform only the conservative static analysis (Kumar, Kushwaha and Sinha, 2018) due to its simplicity while it is clearly mentioned in UBC-97 when the height of an irregular building exceeds 65 ft, dynamic analysis must be performed. Hopefully, by this research we can make them understand the significance of using dynamic analysis because it gives lesser values of all critical parameters associated with seismic event (Kakpure and Mundhada, 2017).

In the present work, a high-rise building comprising of 22 storeys with 04 basements located at sector F9 Islamabad (Seismic zone 2B) is analyzed both by Equivalent Static Analysis and Response Spectrum Analysis method using Structural Analysis Software ETABS 2016.

2. METHOD OF ANALYSIS

Linear Static Analysis "Equivalent Lateral Force Method" and linear dynamic analysis "Response Spectrum Analysis" were performed on the structure by ETABS and a comparison is made between the two analysis techniques.

2.1 Equivalent Lateral Force Method

The forces developed under seismic event are dynamic in nature, so the design of buildings for such forces must consider the dynamic nature of forces (Kakpure and Mundhada, 2017). However, for simple and regular structures with small to medium heights static analysis is considered sufficient and allowed by Universal building Code (UBC Code, 1997) and Building Code of Pakistan (BCP, 2007).

Equivalent Lateral force method (ELF) involves the calculation of base shear and its distribution along the height by formulae given in the UBC-97. This is a conservative method used in low to medium height buildings and it assumes same acceleration at every point of the structure during ground motion (Kumar, Kushwaha and Sinha, 2018). In ETABS the first step was to model the building. After modelling, load patterns and load combinations were defined. Loads are applied on the building as per UBC-97. As the building is in Islamabad seismic zone 2-B, so peak ground acceleration of 0.2g is used as per Building Code of Pakistan. Lastly, in the analysis menu from the "set load cases to run" run linear static for Equivalent Lateral Force Method.

2.2 Response Spectrum Analysis

Response Spectrum Analysis is a linear dynamic analysis which takes into consideration the dynamic nature of earthquake forces. An earthquake vibrates the ground below the building due to which vibrations are induced in the building. Mode shape is the set of relative nodal displacements for a particular node of free vibration for a specific natural frequency. Each mode shape is associated with a unique frequency of vibration, time period and participating mass. Only those mode shapes are considered in which the participating mass is greater than 90% (UBC Code, 1997). A plot between peak ground acceleration and the time period is obtained which is called Response Spectrum. Equivalent Lateral force is a conservative approach as compare to Response Spectrum analysis because it depends only on the height and weight of the building irrespective of the different vibrations induced in the building (Kumar, Kushwaha and Sinha, 2018). Response Spectrum Analysis is more realistic determination of seismic forces as compared to ELF and we must perform Dynamic analysis when an irregular building height reaches 65 ft (UBC Code, 1997). It considers each mode of vibration along with its time period. For each time period and damping ratio definite spectral acceleration is obtained. Stress resultants are obtained by

displacement response. The final response envelop is obtained by adding stress resultants with some combination rule, complete quadratic combination for this work (Nawy, 2000).
In ETABS to perform Response Spectrum Analysis, first we must perform static analysis, because modelling of the structure is same for both the analysis, moreover base shear from Equivalent Lateral Force method is used in adjusting scale factor for Response Spectrum Analysis. It all starts with defining a Response Spectrum function and select UBC-97. Put 5% damping and assign seismic coefficients of Ca = 0.28 and Cv =0.40 because of seismic zone 2-B. Define a response spectrum load case both in X and Y direction, adjust the initial scale factor (scale factor = gxR/I) and select complete quadratic combination (CQC) as a directional combination method. Modify the modal case and put maximum number of modes equal to 66, as the number of floors is 22. Lastly, in the analysis menu from the “set load cases to run” run Model-eigen along with linear static for Response Spectrum Analysis.

3. MODELLING AND ANALYSIS

The building selected for the present work is Fortune One Tower located in Sector F9 Islamabad having 22 storeys with four basements. Islamabad comes under seismic zone 2B with peak ground acceleration between 0.16 to 0.24g (BCP, 2007). As per seismic zone requirement Intermediate Moment Resisting frames are used in the building. The dimension of the building in X-Y plan is 119′x 99′. Grades of concrete used were 4000psi and 6000psi. Concrete with 4000psi compressive strength is used in beams and slabs while 6000psi concrete is used in columns and shear walls. Grade 60 steel rebar is used throughout the building. The building comprises of 04 basements, lower three basements are used for car parking and the lower ground floor/basement 01 is used as a super store. Ground floor to 3rd floor is used for commercial purposes with MEP and Mezzanine floor. The rest of the floors consist of residential apartments with penthouse at the top. The height of the floors varies from 10ft, 10.5ft, 10.75ft, 11.75ft and a maximum of 13.5 ft at the ground floor. Slabs of 6in, 7in, 8in and 9inch were used. Beams of various dimension 12’’x6”,12”x15”, 12”x18”, 12”x24”, 12”x30”, 12”x42”, 10”x51” etc. were used. Columns sizes mainly used were 48”x18’,36”x24”, 36”x36”, 18”x30”, 24”x30”, 24”x36”, 24”x30” etc. as per requirement. The building contains a central lift with shear walls around it. Retaining wall is provided around four sides of basement. The soil properties are not known in sufficient detail to determine soil profile type, so type S_D is used as per UBC-97 Section 1636.2. The modelling of this building in ETABS is done by grid formation, assigning of number of stories, materials definition, defining of section properties and the assignment of these defined sections to their positions. Further, stiffness modifiers, Diaphragm, floor meshing, End length offsets, load patterns, load combinations, mass source, P-delta effect, assigning of support to the base and finally occupancy and seismic loads were assigned to the building according of BCP and UBC-97. The ETABS generated 3D model of the building is shown in figure 1.

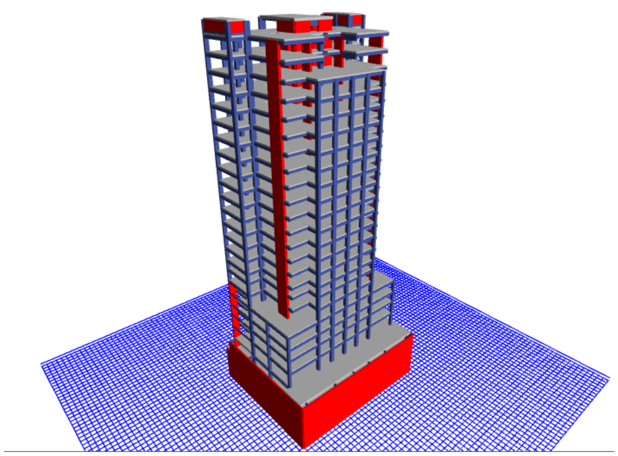


Figure 19: 3D Model of the Building in ETABS

4. RESULTS AND DISCUSSION

The reinforced concrete building under study is analyzed both statically and dynamically by ETABS 2016. The graphical results along with explanation of all critical parameters such as Storey Displacement, Story Drift, axial forces in columns and bending moments in beams are presented below:

4.1 Storey Drifts

Storey Drift is the relative lateral displacement of a floor with respect to the floor above or below it. When this difference of lateral displacement is divided by storey height, it yields storey drift ratio, which is one of the critical parameters in seismic design of high-rise buildings. Maximum inelastic response for buildings ($\Delta M = 0.7R\Delta S$) with time period less than 0.70 seconds shall be less than 2.5% of the height of the building while it is 2% for buildings with time period greater or equal to 0.70 seconds. Table 1 shows the variation of storey drifts along the height of the building followed by its graphical illustration.

Table 6: Comparison of Storey Drifts

Storey	Storey No.	Elevation	Location	EQX (in)	EQY (in)	RSX (in)	RSY (in)
Top Roof	27	232.25	Top	0.001168	0.001058	0.000839	0.000598
Roof	26	223.75	Top	0.001245	0.001155	0.000935	0.000847
18th Floor	25	213.25	Top	0.001292	0.001319	0.000996	0.000945
17th Floor	24	202.75	Top	0.001326	0.001507	0.001049	0.001149
16th Floor	23	192.25	Top	0.001351	0.001684	0.001094	0.001344
15th Floor	22	181.75	Top	0.001365	0.001843	0.001124	0.00152
14th Floor	21	171.25	Top	0.001372	0.001985	0.001141	0.001673
13th Floor	20	160.75	Top	0.001404	0.002109	0.001152	0.001801
12th Floor	19	150.25	Top	0.001423	0.002216	0.001172	0.001904
11th Floor	18	139.75	Top	0.001435	0.002301	0.001183	0.001986
10th Floor	17	129.25	Top	0.001447	0.002359	0.001191	0.002047
9th Floor	16	118.75	Top	0.001453	0.002389	0.001194	0.002087
8th Floor	15	108.25	Top	0.00145	0.002392	0.001189	0.002107
7th Floor	14	97.75	Top	0.001437	0.002375	0.001177	0.002112
6th Floor	13	87.25	Top	0.001416	0.002332	0.00116	0.002098
5th Floor	12	76.75	Top	0.001383	0.002259	0.001137	0.002062
4rth Floor	11	66.25	Top	0.001344	0.002165	0.001109	0.002005
MEP Floor	10	58.25	Top	0.001287	0.002035	0.001069	0.001921
3rd Floor	9	47.5	Top	0.001229	0.001904	0.001029	0.001837
2nd Floor	8	36.75	Top	0.001141	0.001712	0.000963	0.001685
Ist Floor	7	26	Top	0.00104	0.00149	0.000886	0.001497
Mezzanine Floor	6	16.25	Top	0.000902	0.001213	0.000777	0.001238
Ground Floor	5	4.5	Top	0.000668	0.000755	0.000572	0.000749
Basement 01	4	-9	Top	0.000376	0.000231	0.000315	0.000171
Basement 02	3	-19	Top	0.000107	0.000075	0.00009	0.000052
Basement 03	2	-29	Top	0.000039	0.000036	0.000035	0.000029
Basement 04	1	-39	Top	0.000037	0.000022	0.00003	0.00002
Base	0	-49	Top	0	0	0	0

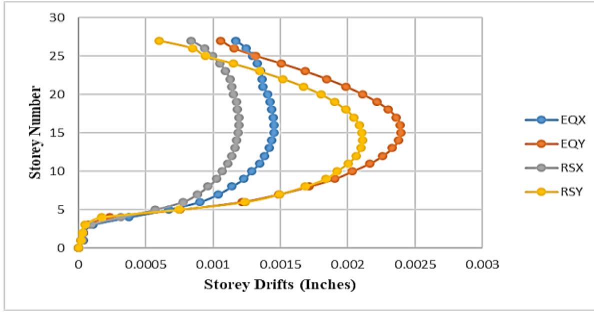


Figure 20: Comparison of Storey Drifts

From Figure 2, it is observed that Equivalent Lateral Force (ELF) gives 18% greater values of maximum story drift in the X-direction (EQX) than Response Spectrum Analysis (RSX). Correspondingly, maximum story drift in Response Spectrum Analysis (RSY) in the Y-direction is 12% less than Static Analysis (EQX).

4.2 Storey Displacements

Storey displacement is a measure of displacement of a story with respect to the base of the building. The values of storey displacement increases from the base to the top of the building. The variation of storey displacements at different floor levels is shown in Table 2 followed by graphical illustration.

Table 7: Comparison of Storey Displacements

Storey	Storey No.	Elevation	Location	EQX (in)	EQY (in)	RSX (in)	RSY (in)
Top Roof	27	232.25	Top	3.518743	5.290068	2.713696	4.437741
Roof	26	223.75	Top	3.520021	5.248363	2.71274	4.416654
18th Floor	25	213.25	Top	3.387428	5.105232	2.674771	4.335152
17th Floor	24	202.75	Top	3.293805	4.938997	2.608004	4.229625
16th Floor	23	192.25	Top	3.137352	4.74908	2.516898	4.099304
15th Floor	22	181.75	Top	2.974967	4.536924	2.392562	3.945521
14th Floor	21	171.25	Top	2.807033	4.304706	2.262362	3.770346
13th Floor	20	160.75	Top	2.634105	4.054622	2.127143	3.576197
12th Floor	19	150.25	Top	2.457197	3.788856	1.988048	3.365585
11th Floor	18	139.75	Top	2.277902	3.509683	1.846477	3.141073
10th Floor	17	129.25	Top	2.097064	3.219819	1.703299	2.904993
9th Floor	16	118.75	Top	1.914744	2.92257	1.558853	2.659572
8th Floor	15	108.25	Top	1.731625	2.621618	1.41371	2.407235
7th Floor	14	97.75	Top	1.548884	2.32018	1.268696	2.150455
6th Floor	13	87.25	Top	1.367779	2.020932	1.124627	1.891367
5th Floor	12	76.75	Top	1.189424	1.727107	0.982157	1.632338
4rth Floor	11	66.25	Top	1.015138	1.442486	0.842114	1.37645
MEP Floor	10	58.25	Top	0.88611	1.234633	0.737721	1.186145
3rd Floor	9	47.5	Top	0.718244	0.974224	0.600728	0.946887
2nd Floor	8	36.75	Top	0.559639	0.728556	0.469948	0.711163
Ist Floor	7	26	Top	0.412447	0.507729	0.347263	0.494585
Mezzanine Floor	6	16.25	Top	0.290822	0.333414	0.244716	0.319951
Ground Floor	5	4.5	Top	0.163611	0.162324	0.136301	0.146472
Basement 01	4	-9	Top	0.06108	0.040754	0.051759	0.029504
Basement 02	3	-19	Top	0.020442	0.013681	0.016843	0.010447
Basement 03	2	-29	Top	0.007545	0.007012	0.006714	0.005879
Basement 04	1	-39	Top	0.004425	0.002691	0.003574	0.002421
Base	0	-49	Top	0	0	0	0

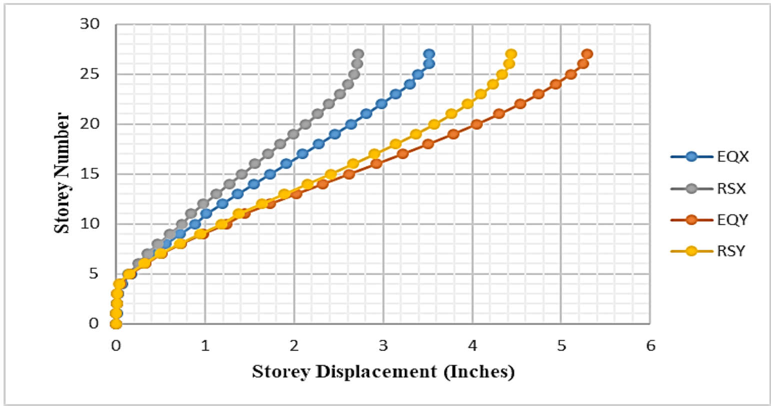


Figure 21: Comparison of Storey Displacements

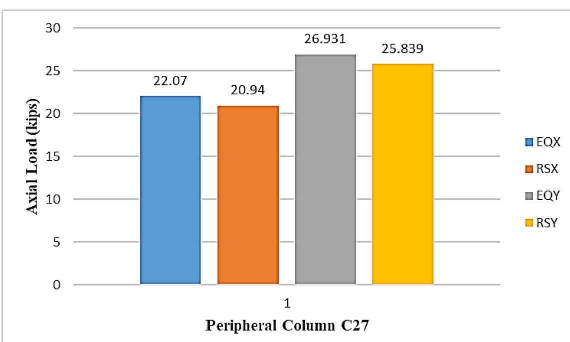
From figure 3, it is observed that maximum Storey Displacements in Equivalent Lateral Force Method (EQX) in X-direction is 23% greater than Response Spectrum Analysis (RSX). Similarly, maximum Storey Displacement in Y-direction in Response Spectrum Analysis (RSY) is 17% less than its value in Equivalent Lateral Force Analysis (EQY).

4.3 Maximum Axial Load in Columns

Three columns of different locations, Interior, peripheral and corner columns are selected for this comparison to cover all the necessary positions.

4.3.1 Peripheral column

From the bar chart shown in figure 4, it is obvious that axial load of peripheral column in Response Spectrum Analysis (RSX) is 5% less than Equivalent Lateral Force Analysis (EQX) in the X-direction. In the Y-direction, maximum axial load EQY is 4% greater than RSY.



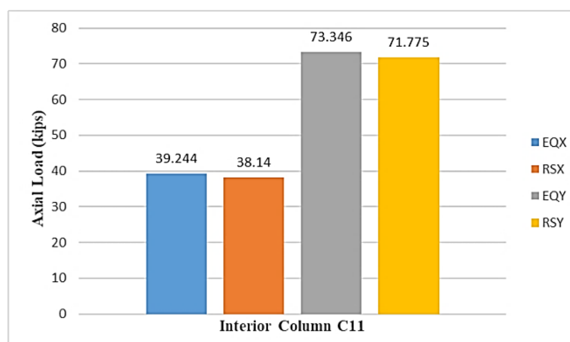


Figure 22: Maximum Axial load at Peripheral Column *Figure 23: Maximum Axial Load in Interior Column*

4.3.2 Interior column

From the bar chart shown in figure 5, it is observed that Response Spectrum Analysis (RSX and RSY) and Equivalent Lateral Force Analysis (EQX and EQY) values are quite the same in their respective X and Y directions. Maximum loads due to ELF are only 2% greater than Response Spectrum Analysis (RSA).

4.3.3 Corner column

From the bar chart shown in figure 6, it is observed that axial load of corner column in Response Spectrum Analysis (RSX) are 7% less than Equivalent Lateral Force Method (EQX) in the X-direction. In the Y-direction, Static Analysis (EQY) are 5% greater than Dynamic Analysis (RSY).

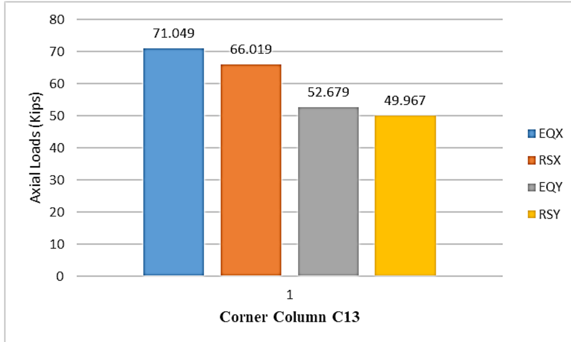


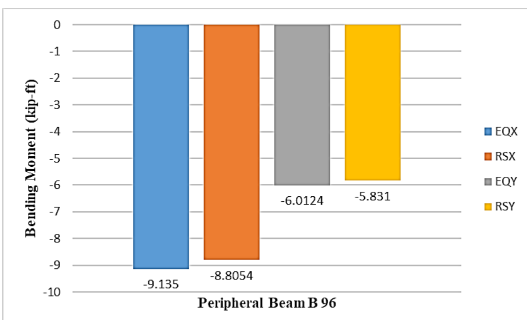
Figure 24: Maximum Axial load at Corner Column

4.4 Comparison of Beam Bending Moments

Three beams of different locations, Interior, peripheral and corner beams are selected for this comparison to cover all the necessary positions.

4.4.1 Maximum bending moment in peripheral beam

From the bar chart shown in figure 7, it is obvious that bending moment of peripheral beam in Dynamic Analysis (RSX) is 4% less than Static Analysis (EQX) in the X-direction. In the Y-direction, maximum bending moment in Static Analysis (EQY) is 3% greater than Dynamic Analysis (RSY).



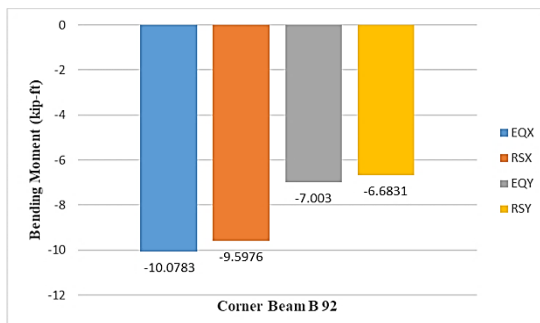


Figure 25: Maximum B.M in Peripheral beam *Figure 26: Maximum bending Moment in End Beam*

4.4.2 Maximum bending moment in end beam

From the bar chart shown in figure 8, it is observed that bending moment of end beam in Response Spectrum Analysis for both directions is 4.5% less than Equivalent Static Analysis in the respective direction.

4.4.2 Maximum bending moment in interior beam

From the bar chart shown in figure 9, it is observed that bending moments in interior beams due to Response Spectrum Analysis RSX and RSY are approximately 3% less than Static Analysis EQX and EQY in both X and Y directions respectively.

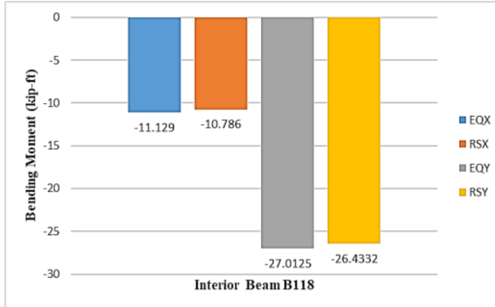


Figure 27: Comparison of Maximum Bending Moment in Interior Beam

5. Conclusions:

- In Dynamic Analysis values of Storey Drifts are 18% less than Static Analysis.
- Storey displacement increases gradually along the height of the building with highest displacement on the top of the building in both X and Y directions.
- Dynamic Analysis gives 23% less values for Storey Displacement as compared to Static Analysis.
- Axial loads in Dynamic Analysis at the Corner and Peripheral columns are 4 to 7% less than Static Analysis. However axial load for interior column is not reduced significantly by Dynamic Analysis.
- Bending moments in beams are 3 to 4% less in Dynamic Analysis than Static Analysis.
- All the critical parameters like story drift, story displacement, bending moment in beams and axial load on columns shows significant low values in Dynamic Seismic Analysis as compared to Static Seismic Analysis. The precise estimation of seismic forces and structural response in dynamic analysis is accounted for reduction in all critical parameters. So, it is highly recommended to use Dynamic Analysis in high-rise buildings.
- This work focuses on Linear Static and Linear Dynamic Analysis and can be improved by performing Non-linear Static Push Over Analysis and Non-linear Dynamic Time History Analysis.

6. References:

Avouac, J. *et al.* (2006) 'The 2005 , M w 7 . 6 Kashmir earthquake : Sub-pixel correlation of ASTER images and seismic waveforms analysis', 249, pp. 514–528. doi: 10.1016/j.epsl.2006.06.025.

Code, U. B. (1997a) 'UBC-97', in *Structural engineering design provisions. International conference of building officials, Whittier, California*.

Code, U. B. (1997b) 'UBC. 1997', in *International Conference of Building Officials, Uniform Building Code, Whittier, California*.

Durrani, A. J. *et al.* (2005) 'the Kashmir Earthquake of October 8 , 2005 a Quicklook Report', *Earthquake*.

Ibrahim, E. (2007) 'High- Rise Buildings – Needs & Impacts', pp. 1998–2008.

Kakpure, G. G. and Mundhada, D. A. R. (2017) 'Comparative Study of Static and Dynamic Seismic Analysis of Multistoried RCC Buildings by ETAB', *International Journal of Engineering Research and Applications*, 07(05), pp. 06–10. doi: 10.9790/9622-0705050610.

Kumar, N., Kushwaha, D. and Sinha, R. (2018) 'Earthquake Resistant Design & Comparison of (G + 7) OMRF Building at Meerut by Using Equivalent Lateral Force & Response Spectrum Method', 6(Vii), pp. 456–462.

Naseer, A. *et al.* (2010) 'Observed seismic behavior of buildings in Northern Pakistan during the 2005 Kashmir earthquake', *Earthquake Spectra*, 26(2), pp. 425–449. doi: 10.1193/1.3383119.

Nawy, E. (2000) *Reinforced concrete: A fundamental approach*.

Seismic Provisions (2007) 'Building Code of Pakistan (Seismic Provisions 2007)', p. 303.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Improvement in Impact Resistance of GFRP Rebars Reinforced Concrete Wall Panels Using Jute Fibres

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Abstract

Concrete walls act as a shield against blast for the security personals in case of explosion. These walls may or may not sustain the impact of blast and launch debris at high velocity, which needs to be avoided. The goal of this study is to investigate the improvement in impact resistance of GFRP rebars reinforced concrete wall panels using jute fibres. Two prototype wall panels having dimensions of (375 x 375 x 50) mm are prepared using mix design ratio of 1:2:3 and 0.6 w/c ratio. In one panel, plain concrete (PC) is used and taken as reference panel. For other panel, jute fibres with 50 mm length and 5%, by cement mass are added. Specimens are tested against impact loading using modified pendulum impact apparatus. Number of strikes are performed in intervals (each having five strikes). Dynamic properties are investigated as per ASTM C215-02 before any impact, after initial cracking and after failure. A specimen is said to be failed if the spall depth of concrete reaches to 25mm. Damping ratios are determined to study the effect of fibres on improvement of impact resistance. Post impact condition of fibre surface and fibre concrete bond is examined by SEM testing. The impact strength of jute fibre reinforced concrete (JFRC) wall panel came out to be more than PC in terms of initial crack strength and ultimate failure strength. Decrement in dynamic elastic modulus and increment in damping is greater in JFRC as compared to PC. SEM analysis shows fibre fracture and circumferencial cavity formation around fibre due to impact on concrete. Thus, GFRP rebars reinforced concrete wall panels having jute fibres perform better against impact loading.

Keywords

Concrete Walls, Impact Loading, Jute Fibre Reinforced Concrete, Pendulum Impact.

1. Introduction

Buildings can experience extreme dynamic loading in the form of suicide/ grenade explosions. Reinforced concrete walls not only protect the internal facilities from the impact of explosion but also act as a shield during retaliation in case of such events. Reinforced concrete walls exposed to such dynamic loading conditions are supposed to sustain shock and impact due to explosion (Zineddin and Krauthammer, 2007). The impact of an explosion is in inverse relation with its duration. The ultimate resistance of a structural element decreases as the ductility increases in order to safely resist an air blast loading (Yang and Lok, 2007). Impact resistance of a structural element is the property that ensures the safety of element under dynamic loading conditions (Hussain and Ali, 2019). Seica et al. (2019) investigated the effects of blast and impact loading on composite concrete filled hollow structural sections. Small scale dynamic testing and field blast testing was performed and compared with the results of numerical modelling.

In case of explosion of a grenade or a suicide jacket, the fragments or bearing balls disperse and strike with high velocity at the structures nearby. This phenomenon causes the damage of structural element leading to breakup and launch of debris. Wu et al. (2019) investigated the characteristics and scattering pattern of debris in reinforced concrete slabs subjected to blast loading. The impact of explosion also depends on its distance from the aimed structure. An air blast caused by suicide jacket, launch the bearing balls which may or may not directly hit the target. The ability of a structural member to sustain an explosion greatly depends upon the dynamic response characteristics and failure mode of that structure. Chao et al. (2019) performed the experimental and numerical damage assessment of reinforced concrete beams due to explosion by considering the parameters like position, distance and weight of explosives. Mishra and Sharma (2018) reviewed the mechanical properties and impact resistance of ultra-high-performance fibre reinforced concrete (UHPFRC) by considering parameters like energy absorption and effectiveness of hybrid steel fibres and compared it with the properties of reinforced concrete having only single type of fibre. Luccioni et al. (2017) evaluated the performance of high strength concrete reinforced with steel fibres in response to blast loading and found that thickness of flexural cracks, zone of spalling and permanent deflections decreased due to addition of fibres thus improving behavior of concrete significantly against blast loading.

The performance desired from a reinforced concrete wall is to possess enough strength that can sustain a blast load and properties that can restrict the launch of debris after the explosion. To enhance the static and dynamic properties of concrete, fibres can be added with suitable proportions. Islam and Ahmad (2018) evaluated the influence of jute fibre on the properties of concrete. The failure patterns indicated that number of cracks and their width decreased due to inclusion of fibres. The impact strength of fibre reinforced concrete greatly depends upon its toughness i.e. that is the ability of fibre to absorb energy. Ramakrishna and Sundararajan (2005) investigated the impact strength of natural fibre reinforced composites in cement mortar slabs and it was found that impact strength of such composites was significantly greater than those of plain concrete. Sheikh and Kharal (2018) performed compression tests on confined columns reinforced with GFRP rebars. The longitudinal GFRP rebars were able to resist compressive stresses in excess of 700 MPa and lateral GFRP rebars were able to confine concrete more efficiently than steel. Pham and Hao (2016) investigated the axial impact resistance of GFRP and CFRP confined concrete columns. The GFRP showed a significant enhancement in strength and ductility against impact as compared to CFRP.

To the best of author's knowledge, no work has been done to investigate the strength of concrete reinforced with jute fibres and GFRP rebars for its application in walls exposed to impact loading. Thus, effects of jute fibres addition in GFRP rebars reinforced concrete walls needs to be investigated under impact loading.

2. Experimental Procedure

2.1 Raw Materials

Ordinary Portland Cement, locally available fine and coarse aggregates, water, GFRP rebars and jute fibres are used for the preparation of jute fibre reinforced concrete (JFRC) having GFRP rebars. The length of each jute fibre is 50 mm. θ6 mm GFRP rebars are used.

2.2 Mix Design and Casting Procedures

For the preparation of JFRC, the mix design ratio used is 1:2:3 (Cement: Sand: Aggregate) with 0.6 water cement ratio. Jute fibres are added 5% by mass of cement. Firstly, one third of the coarse aggregates and fibres are added in the drum mixer with three quarters of water and then mixer is rotated for two minutes. Then two third fine aggregates are added and mixer is rotated for another two minutes duration. Then rest of the materials are added and mixer is rotated for three minutes. Slump test is performed to investigate the workability of JFRC which came out to be 36 mm as compared to 58 mm of PC, as per specifications of ASTM C143 / C143M-15a.

2.3 Specimens

Cylinders having dimensions (100 x 200) mm and beamlets having dimensions (100 x 100 x 450) mm are prepared for the investigation of mechanical properties as per ASTM standards. Prototype walls, having dimensions (375 x 375 x 50) mm with a mesh of 6 x 6 GFRP rebars, are prepared for impact test.

2.4 Mechanical Properties

Compression, splitting tension and flexural tests are performed in servo hydraulic testing machine. The strengths of PC and JFRC came out to be 13.11 MPa and 11.33 MPa against compressive loading, 8.69 MPa and 5.94 MPa against splitting tensile load, 4.23 MPa and 2.3 MPa against flexural loading, respectively.

2.5 Impact Test

To perform impact test on PC and JFRC wall panels having GFRP rebars, walls are fixed one after the other in the modified pendulum impact apparatus and then strikes of hammer are performed by releasing the hammer from an angle of 90^o. Strikes are performed till failure of specimen (A specimen is said to be failed if the spall depth of concrete reaches to 25mm). Modified pendulum impact apparatus is shown in Figure 1. The resonant frequencies of the wall panels are determined and dynamic elastic modulus is calculated as per specifications of ASTM C215-02. The SEM analysis is also performed on the fractured walls to study the post impact condition of fibres.

Figure 28: Modified Pendulum Impact Apparatus

3. Results and Analysis

3.1 Impact Test Behavior

Impact test is performed for both PC and JFRC wall panels having GFRP rebars. Figure 2a shows the cracking pattern of PC and JFRC wall panels at initial crack strength (ICS) and ultimate failure strength (UFS). It can be observed that JFRC panel has experienced more damage than PC at ICS and UFS. At

ICS, a minor hair line crack is visible in PC panel at the point of impact while on the other hand an extended hair line crack is visible in JFRC slab which shows the brittleness of PC panel as compared to JFRC panel.

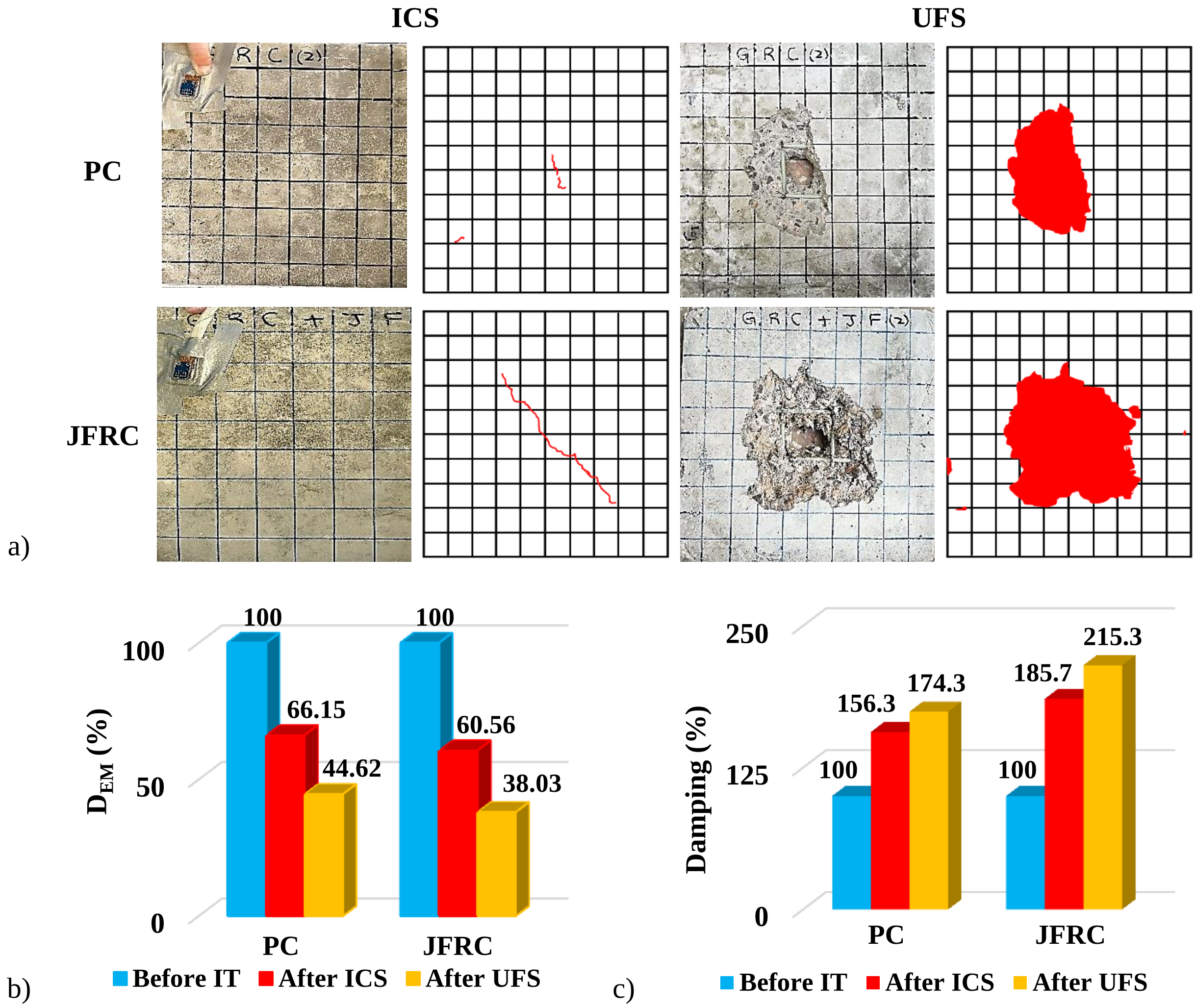

Figure 2: Impact Behavior of PC and JFRC having GFRP rebars; a) Cracking Patterns, b) Percentage Decrement in Dynamic Elastic Modulus (D_{EM}) and Percentage Increment in Damping.

Figure 2b shows that after ICS the dynamic elastic modulus has decreased more in case of JFRC as compared to PC. Similarly, after UFS the dynamic elastic modulus of JFRC has decreased to 38% while on the other hand dynamic elastic modulus of PC has decreased to a little less than half of its initial value before impact test (IT). The behavior of both PC and JFRC in terms of damping seems quite clear up to ICS and UFS that shows the increment in damping of JFRC relatively more than that of PC as shown in Figure 2c.

3.2 Impact Test Parameters

Table 1 shows the impact test parameters; mass of hammer (M_I), impact force generated (F_I), number of strikes till ICS, number of strikes up to UFS and extension of spalling mass from point of impact (S_L). It can be observed that JFRC has taken 48.8% more number of strikes than that of PC till ICS. Similarly, the number of strikes till ultimate failure of JFRC are 57.4% more than that of PC. This indicates the energy absorption capacity of JFRC against impact loading. The point to be noted is the spalling mass of concrete which is 4.1% in PC and 7.4% in JFRC after impact test and its extension from the point of impact which is also greater in case of JFRC than PC.

Table 8: Impact Test Parameters

Specimen	M_I (kg)	F_I (N)	ICS (strikes)	UFS (strikes)	S_L (mm)
PC + GFRP	2.215	5.04	43	75	71.6 ± 9.6
JFRC + GFRP	2.215	5.04	84	176	78.1 ± 15.6

4. Discussion

The impact energy absorbing capacity of fibre reinforced concrete depends upon the tensile strength of fibre and fibre concrete bond strength to withstand the concrete fragments against spalling. Figure 3a shows the SEM view of post impact fibre concrete matrix. It is evident that the striking impact force has weakened the concrete fibre bond causing circumferential cavity formation around fibre. This is due to the movement of fibre during utilization of its strength against pull out.

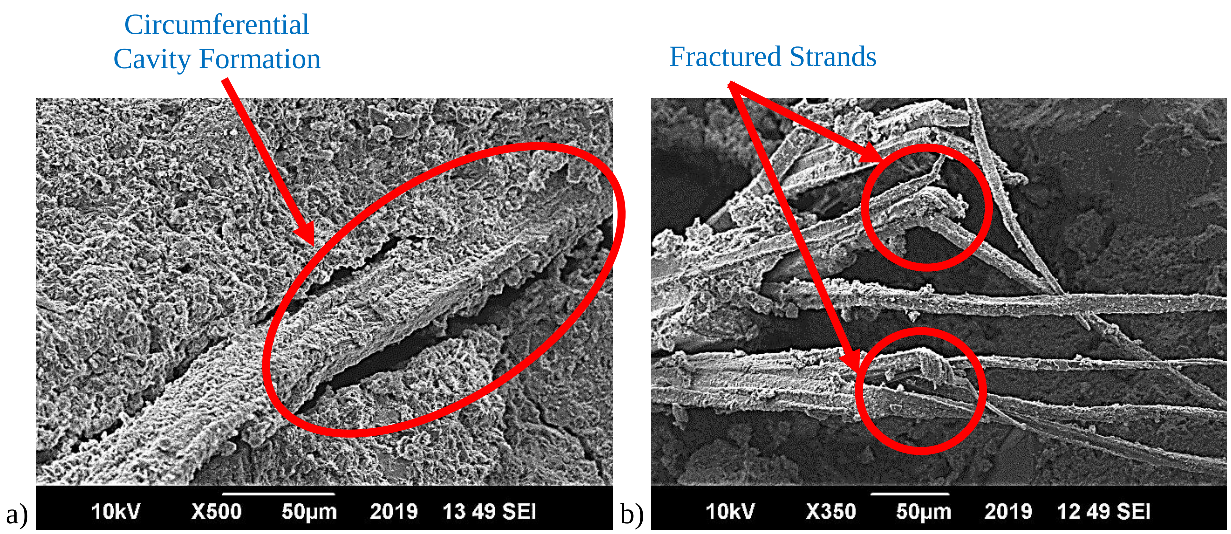

Figure 3: SEM Images of JFRC panel; a) Post Impact Fibre Concrete Bond, b) Damaged Fibres.

Another SEM image is shown in Figure 3b that elaborates the fibre damage specifically at location of impact strikes. The roughened surface of fibre strands and fractured condition shows the strong tensile strength of fibre which has caused the pullout of fibre from concrete.

5. Conclusions and Recommendations

The study investigates the impact behaviour of JFRC and PC wall panels having GFRP rebars and mix design ratio 1:2:4 with jute fibres 5% by mass of concrete using modified pendulum impact apparatus. The reduction in dynamic elastic modulus and damping of JFRC turned out to be less as compared to PC. Following are the conclusions drawn from the conducted research:

- Addition of jute fibres enhances the energy absorption capacity of concrete against impact loading.
- Tensile strength of jute fibres plays vital role in holding concrete fragments provided strong bond between fibres and concrete matrix.

The performance of JFRC in GFRP rebars reinforced wall panel has shown a noticeable impact resistance. Thus, future researches should be oriented towards the durability of jute fibres in concrete.

6. Acknowledgements

The author would like to thank Mr. M. Nadeem and Mr. M. Junaid for his technical assistance during operating mixer and testing. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

7. References

Hussain, T., & Ali, M. (2019). Improving the impact resistance and dynamic properties of jute fiber reinforced concrete for rebars design by considering tension zone of FRC. *Construction and Building Materials*, Vol. 213, pp. 592-607.

Islam, M. S., and Ahmed, S. J. (2018). Influence of jute fiber on concrete properties. *Construction and Building Materials*, Vol. 189, pp. 768-776.

Luccioni, B., Isla, F., Codina, R., Ambrosini, D., Zerbino, R., Giaccio, G., & Torrijos, M. C. (2017). Effect of steel fibers on static and blast response of high strength concrete. *International Journal of Impact Engineering*, Vol. 107, pp. 23-37.

Mishra, S., & Sharma, H. K. (2018). Impact resistance and mechanical properties of UHPFRC. *Iranian Journal of Science and Technology, Transactions of Civil Engineering*, pp. 1-10.

Pham, T. M., & Hao, H. (2016). Axial impact resistance of FRP-confined concrete. *Journal of Composites for Construction*, Vol. 21, No. 2, pp. 1-11.

Ramakrishna, G., & Sundararajan, T. (2005). Impact strength of a few natural fibre reinforced cement mortar slabs: a comparative study. *Cement and concrete composites*, Vol. 27, No. 5, pp. 547-553.

Seica, M. V., Packer, J. A., & Yankelevsky, D. Z. (2019). Blast and impact loading effects on glass and steel elements and materials. *Thin-Walled Structures*, Vol. 134, pp. 384-394.

Sheikh, S. A., & Kharal, Z. (2018). Replacement of steel with GFRP for sustainable reinforced concrete. *Construction and Building Materials*, Vol. 160, pp. 767-774.

Shu-Chao, L. S., Dan, L. and Bo, Y. (2019). Experimental study and numerical simulation on damage assessment of reinforced concrete beams. *International Journal of Impact Engineering*, Vol. 132, pp. 1-15.

Wu, Z., Zhang, P., Fan, L., & Liu, Q. (2019). Debris characteristics and scattering pattern analysis of reinforced concrete slabs subjected to internal blast loads–a numerical study. *International Journal of Impact Engineering*, Vol. 131, pp. 1-16.

Yang, G., & Lok, T. S. (2007). Analysis of RC structures subjected to air-blast loading accounting for strain rate effect of steel reinforcement. *International Journal of Impact Engineering*, Vol. 34, No. 12, pp. 1924-1935.

Zineddin, M., & Krauthammer, T. (2007). Dynamic response and behavior of reinforced concrete slabs under impact loading. *International Journal of Impact Engineering*, Vol. 34, No. 9, pp. 1517-1534.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Numerical Investigation of GFRP Reinforced Non-Circular Concrete Column with Fiber-glass Grating Mesh (FGM) Ties

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Abstract
Glass Fiber Reinforced Polymer (GFRP) has excellent corrosion resistant, high strength-to-weight properties, in non-conductive and non-magnetic, making it an ideal substitute to steel reinforcement for projects in corrosive environments (i.e. seashore structure). However, due to its brittle nature, its use as stirrup is the leading issue in GFRP in non-circular reinforced concrete (RC) columns. As stirrups slip over one another at the corner under high axial loading condition. For this purpose, in this study the most unique aspect of 38 mm manufactured fiber-glass grating mesh (FGM) is used as the substitute for transverse reinforcement. Due to uniform construction of FGM, provides the excellent bi-mechanical properties in non- circular shape RC members under the ultimate limit scale (ULS). The present work aims to investigate the effect of FGM as transverse confinement in non-circular RC column by using Non-Linear Finite Element Analysis (NLFEA) tool i.e.; ABAQUS. The present study discussed the problem associated with the numerical modelling of FGM in RC column, like the Concrete Damaged Plasticity (CPD) Model, dilation angle, viscosity parameter, different base feature for GFRP to achieve better results. The results show that the selected NLFEA calibrated parameters predict the behaviour of RC column with FGM at ULS, very close to its experimental results.

Keywords
GFRP, FGM, FEM, ABAQUS

1. Introduction

Corrosion causes the reinforcement to expand, applying pressure onto the surrounding concrete. These tensile stresses can cause spalling and cracking of concrete, accelerating deterioration of the remaining reinforcement and reducing the capacity of the structure (Šavija, Luković et al. 2013). Glass Fiber Reinforced Polymer reinforcement has become the popular substitute to conventional steel reinforcement in corrosive environments. The replacement of steel with Fiber Reinforced Polymer composites such as GFRP bars could be useful in saving the immense repair costs for durability and long-term serviceability of structures (Prachasaree, Piriyakootorn et al. 2015). Furthermore, the retrofitting of old or damaged-reinforced concrete buildings have been done by using GFRP bars and sheets (Bank 2006). GFRP bars are glass fiber composite structural sections, completely resistant to corrosion, nonmagnetic and electrically insulating and have high strength properties (Raza and Ahmad 2019). GFRP sections can be moulded into shapes and sizes matching structural steel standards, making it completely customizable dependent on application. The longitudinal GFRP reinforcement contributes 10-15 % of a columns compressive capacity and should not be ignored when calculating load bearing capacities (De Luca, Matta et al. 2010, Tobbi, Farghaly et al. 2012, Afifi, Mohamed et al. 2014, Tobbi, Farghaly et al. 2014). Also transverse reinforcement configuration and spacing predominantly affected ductility rather than strength, with greater confinements giving the most efficient results (Tobbi, Farghaly et al. 2012, Tavassoli 2013, Afifi, Mohamed et al. 2014, Tobbi, Farghaly et al. 2014, Prachasaree, Piriyakootorn et al. 2015, Hadi, Karim et al. 2016). Efficient stirrup design is a leading issue in present GFRP reinforced columns, as transverse reinforcement heavily affects the ductility of failure. Also, current methods of producing GFRP reinforcement stirrups require permanent, time-consuming processes. Stirrups must be ordered months in advance with no variability in design between ordering and construction. Studies have attempted to partly solve this problem by using multiple C-shaped GFRP sections overlapped to form a stirrup. However, with this method, sections slip past one another under high axial load, reducing core confinement and causing longitudinal reinforcement failure (Tobbi, Farghaly et al. 2014). In an attempt to solve this issue, this experiment use fiber-glass grating mesh (FGM) ties as transverse reinforcement and GFRP plate rebar as longitudinal reinforcement.

Furthermore, massive work has already been done experimentally on behaviour of longitudinal GFRP reinforced concrete members as compared to numerical investigation. The FEM of reinforced concrete exterior and interior beam-column joints was conducted by using the concrete damaged plasticity model in ABAQUS. The crack patterns, deformations and joint shear capacity were studied concluding that the FEA and experimental work are in good agreement (Najafgholipour, Dehghan et al. 2017). (Prachasaree, Piriyakootorn et al. 2015) studied the effect of different types of stirrups in GFRP RC columns resulting in the increased confining pressure because of increased inelastic deformation and concrete strength. (Nunes, Correia et al. 2013) conduct the experimental as well as numerical investigation under eccentric loading on pultruded GFRP columns with the conclusion that when load is applied inside the kern boundary, 40% reduction was observed in load-carrying capacity of column.

The numerical behaviour of GFRP reinforced compression members is not much defined by using FEA software ABAQUS, and there is no numerical study performed using FGM ties as transverse reinforcement. Therefore, a numerical investigation is required for GFRP reinforcement in non-circular columns with FGM ties to study the effect of critical parameters on axial capacity of compression member. The key objective of this research study is proposing a non-linear finite element analysis model in ABAQUS which presents the behaviour of GFRP RC column with FGM ties accurately and validate the model against the load-deflection curve of experimental tests. The significance of this work is to use the proposed model for the analysis of different column parameters.

2. Fiber-glass Grating Mesh (FGM)

The FRP Grating Mesh is composite material produced of thermosetting plastic resin strengthened by fine fibers of glass along with the use of additives (Composites 2019). It is fabricated through moulding process, where the resin and glass-fibers are weaved and layered over a large square mesh pattern. This method was chosen over the pultrusion process due to the excellent bi-mechanical properties of the material that can’t be achieved in the pultrusion process. As this material is typically used for flooring and manufactured in large panels, the ties were cut to the appropriate size by first being cut into large stirrups and then into the individual ties. These are ideal and safe having long maintenance free life for chemically corrosive environment (B. Devender 2013). Properties such as fire retardancy, corrosion resistance, maintenance-free performance, ease of fabrication and installation, light weight, impact resistance, uniform appearance, cost effectiveness, non-conductivity, and bi-directional load bearing have all made moulded gratings to be more desirable.

Figure 29(a) Moulding process for the FRP grating Mesh (b) Typical FRP Grating Mesh floor panel. Photo courtesy of Perma Composites (Perth).

Instead of constructing large floor panels and cutting it into the individual ties, each tie could be individually made by weaving and layering the fibers through the same moulding process. This would ensure that the fibers remain continuous and in a 'close loop', as opposed to being cut off and having its strength reduced. Having the grid-like structure means that longer fibers can be used, which will greatly increase the strength of ties.

3. Finite Element Modelling

3.1 General Methodology

ABAQUS, a commercial software was used to model the GFRP RC non-circular column with FGM ties discussing the boundary conditions, concrete damaged inelasticity model, and different material and geometric parameters. The ABAQUS model was then calibrated for viscosity parameter and dilation angle
of CDP model, and different base feature for GFRP, to achieve results that are in good comparison with the experimental work done by Mohamed Elchalakani. The GFRP RC non-circular column section view for 18x6 (mm) Plate rebar and side view for concentric test are shown in Figure 2.

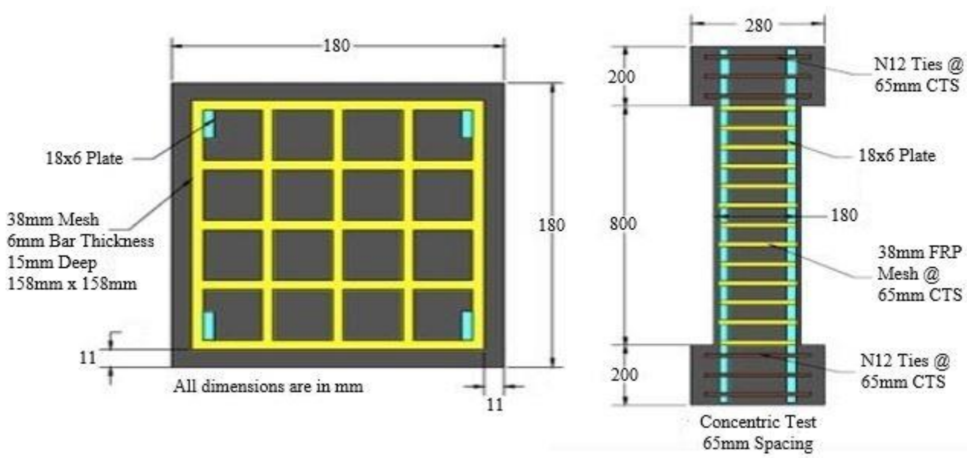

Figure 30. The GFRP RC non-circular column section and side view

3.2 Boundary Conditions and Loading

The GFRP reinforced non-circular column was fixed at bottom and set fee at top under axial loading. The embedded region constraint was used for defining the bonding of reinforcing rebars i.e., steel (in column cap), GFRP 18x6 Plate rebars and FGM ties. The displacement control technique was used for determining the load-deflection history of GFRP square column up to failure. The static monotonic load in form of displacement of 10mm was applied at top center of column specimen. For uniform distribution of load, at both ends of specimen steel plates were attached using tie-constraint. Different column parameters are given in Table 1. The surfaces of concrete and steel plates were interacted using master and slave surface concept. The bottom surfaces of top steel plate and column were taken as master surfaces and top surfaces of bottom steel plate and of column were taken as slave surfaces. The loading increments defined in Step Module of ABAQUS and other concrete properties were given in Table 1.

Table 1. Different column parameters and properties of concrete and loading increments.

Column Parameter	Value	Parameters	Value
Area	32,400 (mm^2)	Concrete Density	2.4E-009 (ton/mm^3)
Moment of Inertia	87,480,000 (mm^4)	Concrete Compressive Strength	35 (MPa)
Slenderness Ratio	23.09	Poisson's ratio, ν	0.2
Radius of Gyration	51.96 (mm)	Young's modulus, E_c	27805.575 (N/mm^2)
Steel Plate Thickness	50(mm)	Concrete cover	11 (mm)
		Initial increment size of loading	0.01
Steel Plate Young's Modulus	200 (GPa)	Max increment size of loading	0.1
		Min increment size of loading	1E-050
Steel Plate Density	7.89×10^{-9} (ton/mm^3)	Number of increments	100000

A GFRP non-circular RC column specimen reinforced with 18x6 (mm) GFRP Plate rebars longitudinally and with FGM ties transversely under axial loading was modelled in ABAQUS. The geometric and the material characteristics of column specimen are given in Table 2.

Table 2. Geometric and material characteristics of column specimen

Test Type	Material	Longitudinal Reinforcement			Transverse Reinforcement				
		No. of Bars	Section	Reinforcement Ratio (%)	Section	Bar Thickness (mm)	Depth (mm)	Dimensions (mm x mm)	Spacing (mm)
Concentric	GFRP	4	18x6 Plate	1.33	38mm Mesh	6	15	158x158	65

The geometry and the modelling details of FEM of GFRP-reinforced column with FGM ties in ABAQUS are shown in Figure 3.

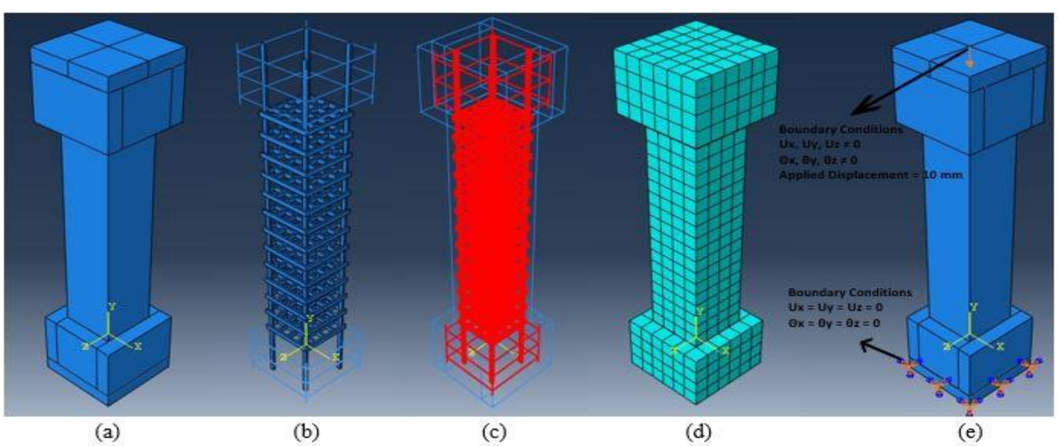

Figure 31. A GFRP reinforced non-circular column with FGM ties (a) column geometry, (b) GFRP 18x6 Plate rebars with FGM ties, (c) embedded region constraint, (d) meshing, (e) top and bottom boundary conditions of column.

3.3 Concrete Damaged Plasticity (CDP) Model

In ABAQUS, three constitutive models including brittle cracking concrete (BCC) model, concrete smeared cracking model (CSCM) and concrete damaged plasticity (CDP) model were used to define the inelastic behaviour of concrete. In CDP model, the tensile behaviour, compressive behaviour, plastic behaviour and concrete damaged mechanism are studied. The convergence of results to accuracy of this model is greater as compared with other models.

3.3.1 Plastic behaviour of concrete

The CDP model of concrete is defined by the required parameters of viscosity, dilation angle, the ratio of compressive stress in the biaxial state to the compressive stress in the uniaxial state (σ_{b0}/σ_{c0}), the plastic potential eccentricity of concrete (ε), and the shape factor (K_c). The recommended CDP model values of the eccentricity, ε, is 0.1, and yield shape surface, K_c, is 2/3 respectively. The value of σ_{b0}/σ_{c0}, is 1.16 by ABAQUS user manual (Systèmes 2010).

3.3.2 Compressive behaviour of concrete

In CDP model, compressive failure of concrete is defined by increased strain and elastic stress by giving increased degree of inelastic strain ε^{in}. According to Eurocode 2, the diagram for concrete's compressive stress-strain is given in figure 4(a). The linear elastic behaviour can be taken up to 0.4 fcm according to Kmiecik and Kaminski (Papanikolaou and Kappos 2007).

3.3.3 Tensile behaviour of concrete

In CDP model, tensile behaviour is defined by using a condition in which a stress-strain relationship at post-failure is used for interaction of concrete with reinforcement, tension stiffening and the strain softening. (Nayal and Rasheed 2006) provide a tension stiffening model, equally applicable for fiber-reinforced polymer and steel reinforced concrete. (Wahalathantri, Thambiratnam et al. 2011) has done some modification in the given by (Nayal and Rasheed 2006) of reinforced concrete by changing the instant drop of the tensile stress-strain curve from ultimate stress σ_{t0} to $0.77\sigma_{t0}$ in place of $0.80\sigma_{to}$ at critical strain ε_{cr}, Figure 4(b).

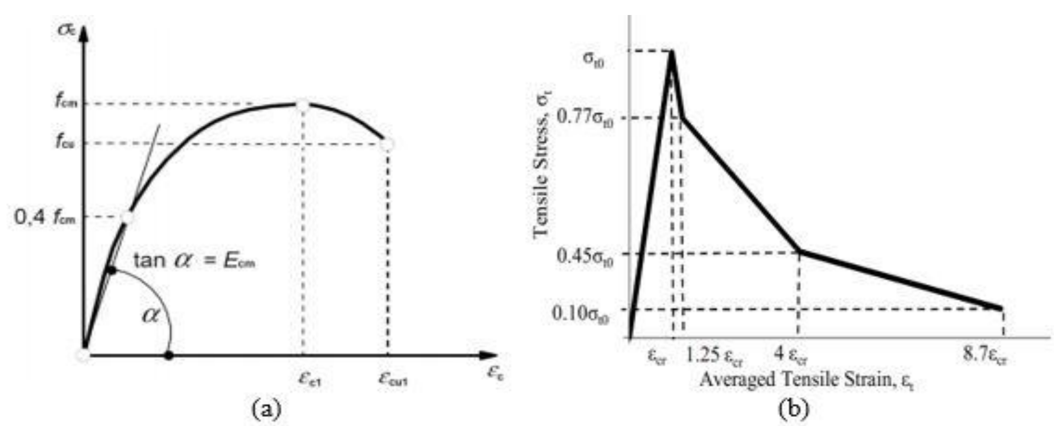

Figure 32. (a) Stress-strain curve for analysis of structures (Eurocode 2), (b) Modified tension stiffening Model

3.4 Modelling of Reinforcing Bars

An isotropic linear elastic behaviour was assumed to model the GFRP bars in ABAQUS, without applying damage criterion up to failure (Ibrahim, Fahmy et al. 2016, Elchalakani, Karrech et al. 2018). The modelling of reinforcing bars needs only four parameters which are elasticity modulus for elastic behaviour and yield strength, Poisson's ratio (ν) and resultant plastic strain for the plastic behaviour of

GFRP and steel rebars. Poisson's ratio (ν) was taken as 0.3 for both GFRP and steel rebars. In Tables 3 the physical properties of used GFRP bars (ARGOSFRP, #136) and steel bars are given.

Table 3. Physical properties of used GFRP and Steel bars

GFRP Bar	Item No.	Bar dimension (mm x mm)	Cross sectional area (mm^2)	Young's modulus (GPa)	Tensile strength (MPa)	Density (ton/mm^3)
	57	18x6	168	48	425	2.1E-009
Steel Bar	Bar No.	Bar diameter (mm)	Cross sectional area (mm^2)	Young's modulus (GPa)	Yield strength (MPa)	Density (ton/mm^3)
	N12	11.8	110	200	505	7.8E-009

3.5 Calibration and Convergence of Model

The FE model was validated and converged by varying the base feature for GFRP, viscosity parameter and dilation to give accurate results and to study their effects on results. The test results of F-PC provided by Mohamed Elchalakani were used as a base for calibration process. The dilation angle is construed as an internal friction angle and material parameter for concrete. In the present study, the influence of the dilation angle on the axial load -deflection curve was examined by taking different values inbetween 30° and 40°. The load-deflection curve gives accurate results to experimental data at 30° by keeping viscosity parameter, 0.01 and mesh size, 45mm shown in Figure 5(a). It can be observed that the effect of different values of dilation angle is very small as compared to viscosity parameter.

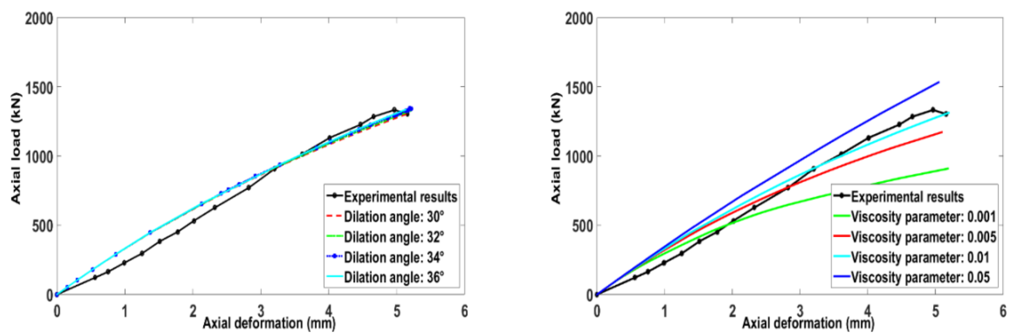

Figure 33. Axial load-axial deflection behaviour of FEA (a)for dilation angle at different values (b)for viscosity parameter using different values.

In Figure 5(b) the load-deflection curve of column specimen shows the effect of varying the viscosity parameter. The time increment size has influenced the value of the viscosity parameter. The small values of viscosity parameter to be used as compared with pseudotime of FEA to achieve better results (Lee and Fenves 1998). The best result was achieved by at value of 0.01 while keeping dilation angle 30° and mesh size 45mm, respectively. In figure 6, the load-deflection curve for column specimen is presented with varying the base feature for GFRP. It was observed that, by using the wire shape for GFRP with cap, the curve is far away from experimental cure and the peak of load-deflection curve moves downward giving more error. The effect of 3D shape with cap and wire shape without cap on load-deflection cure is negligible but the results for 3d shape with cap are closer to experimental curve. Thus, 3D shape for GFRP is best suited while keeping viscosity parameter as 0.01, dilation angle 30° and mesh size 45mm, respectively.

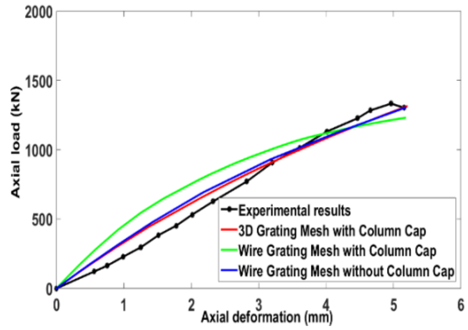

Figure 34. Axial load-axial deflection behaviour of FEA using different base feature for GFRP.

The used values of parameters of CDP model, like the viscosity parameter, shape factor, dilation angle, the ratio of biaxial to uniaxial compressive stresses, and flow potential eccentricity for yielding surface of concrete, are shown in Table 4.

Table 4. Input parameters for CDP model

Plasticity Parameter	Used value
Dilation angle (ψ)	30° (calibrated)
Eccentricity (ε)	0.1 (default)
Stress ratio (σ_{b0}/σ_{c0})	1.16 (default)
Shape factor (K_c)	0.667 (default)
Viscosity parameter	0.01 (calibrated)

4. Conclusions

This research paper has presented the implementation of FEA using ABAQUS to predict behaviour of GFRP in non-circular RC column subjected to axial loading using 18x6 (mm) GFRP Plate rebar as longitudinal reinforcement and 3D FGM ties as transverse reinforcement. The non-linear modelling of reinforced concrete was done using CDP model. In FEM, modelling the materials accurately is challenging, especially the numerical modelling of Concrete and Fiber-glass Grating Mesh ties in concrete structures.

- The key difference in this research is that in this design FRP grating mesh was used as an alternative to conventional GFRP ties, and result shows that this is certainly a feasible and affordable option to consider. The results from this project have shown promising signs and there are design techniques that could be researched in future to build on this. Improving the design of the mesh to see if the ‘pulling out’ of layers of fibers cab be investigated through altering the process in which it is made.
- From the results of FEA model, it was seen that there is an impact of variation of dilation angle (30°, 32°, 34°, 36°), viscosity parameter (0.001, 0.005, 0.01, 0.05), and base feature (3D, wire), for GFRP on FEA results. Also, it was observed from the results that the effect of varying dilation angle, viscosity parameter and base feature were best at 30°, 0.01 and 3D. The dilation angle has very little effect on load-deflection behaviour. Therefore, the viscosity parameter and base feature for GFRP appear to be critical for achieving accurate results.
- The FEA results obtained from ABAQUS in terms of load-deflection curve were found to be consistent with experimental result. The FGM ties used in ABAQUS simulations produced slight overestimations for concentric column. However, the FEA results show the capability of the selected numerical model for prediction of load-deflection behaviour of GFRP-reinforced columns.

References

Afifi, M. Z., H. M. Mohamed and B. Benmokrane (2014). "Axial Capacity of Circular Concrete Columns Reinforced with GFRP Bars and Spirals." Journal of Composites for Construction **18**(1).

B. Devender, A. P., O.Vasudeva Reddy (2013). "Experimental Test on Gfrp Gratings for Mechanical Properties and Chemical Resistance." IOSR Journal of Mechanical and Civil Engineering **8**(6): 34-39.

Bank, L. C. (2006). Composites for construction: structural design with FRP materials, John Wiley & Sons.

Composites, P. (2019). "PermaStruct® FRP Grating Technical Data Guide." **2**.

De Luca, A., F. Matta and A. Nanni (2010). "Behavior of full-scale glass fiber-reinforced polymer reinforced concrete columns under axial load." ACI Structural Journal **107**(5): 589.

Elchalakani, M., A. Karrech, M. Dong, M. M. Ali and B. Yang (2018). Experiments and finite element analysis of GFRP reinforced geopolymer concrete rectangular columns subjected to concentric and eccentric axial loading. Structures, Elsevier.

Hadi, M. N., H. Karim and M. N. Sheikh (2016). "Experimental investigations on circular concrete columns reinforced with GFRP bars and helices under different loading conditions." Journal of Composites for Construction **20**(4): 04016009.

Ibrahim, A. M., M. F. Fahmy and Z. Wu (2016). "3D finite element modeling of bond-controlled behavior of steel and basalt FRP-reinforced concrete square bridge columns under lateral loading." Composite Structures **143**: 33-52.

Lee, J. and G. L. Fenves (1998). "A plastic-damage concrete model for earthquake analysis of dams." Earthquake engineering & structural dynamics **27**(9): 937-956.

Najafgholipour, M., S. Dehghan, A. Dooshabi and A. Niroomandi (2017). "Finite element analysis of reinforced concrete beam-column connections with governing joint shear failure mode." Latin American Journal of Solids and Structures **14**(7): 1200-1225.

Nayal, R. and H. A. Rasheed (2006). "Tension stiffening model for concrete beams reinforced with steel and FRP bars." Journal of Materials in Civil Engineering **18**(6): 831-841.

Nunes, F., M. Correia, J. Correia, N. Silvestre and A. Moreira (2013). "Experimental and numerical study on the structural behavior of eccentrically loaded GFRP columns." Thin-Walled Structures **72**: 175-187.

Papanikolaou, V. K. and A. J. Kappos (2007). "Confinement-sensitive plasticity constitutive model for concrete in triaxial compression." International Journal of Solids and Structures **44**(21): 7021-7048.

Prachasaree, W., S. Piriyakootorn, A. Sangsrijun and S. Limkatanyu (2015). "Behavior and performance of GFRP reinforced concrete columns with various types of stirrups." International Journal of Polymer Science **2015**.

Raza, A. and A. Ahmad (2019). "Numerical investigation of load-carrying capacity of GFRP-reinforced rectangular concrete members using CDP model in ABAQUS." Advances in Civil Engineering **2019**.

Šavija, B., M. Luković, J. Pacheco and E. Schlangen (2013). "Cracking of the concrete cover due to reinforcement corrosion: a two-dimensional lattice model study." Construction and Building Materials **44**: 626-638.

Shokrieh, M. M. and M. Heidari-Rarani (2007). "Analysis of Fibreglass Reinforced Plastic Moulded Grating." Iranian Polymer Journal **16**(8): 511-520.

Systèmes, D. (2010). "Abaqus 6.10: Analysis user's manual." Providence, RI: Dassault Systèmes Simulia Corp.

Tavassoli, A. (2013). Behaviour of GFRP Reinforced Concrete Columns under Combined Axial Load and Flexure.

Tobbi, H., A. S. Farghaly and B. Benmokrane (2012). "Concrete Columns Reinforced Longitudinally and Transversally with Glass Fiber-Reinforced Polymer Bars." ACI Structural Journal **109**(4).

Tobbi, H., A. S. Farghaly and B. Benmokrane (2014). "Behavior of Concentrically Loaded Fiber-Reinforced Polymer Reinforced Concrete Columns with Varying Reinforcement Types and Ratios." ACI Structural Journal **111**(2).

Wahalathantri, B. L., D. Thambiratnam, T. Chan and S. Fawzia (2011). A material model for flexural crack simulation in reinforced concrete elements using ABAQUS. Proceedings of the first international conference on engineering, designing and developing the built environment for sustainable wellbeing, Queensland University of Technology.

11th International Civil Engineering Conference (ICEC-2020) "Integrating Innovation & Sustainability in Civil Engineering" March 13-14, 2020, Karachi, Pakistan.

A review on observed flaws in bricks and their possible sustainable remedies for local construction

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Abstract

Bricks and blocks are the major building construction units. There are some flaws observed in these units and requires an improvement in terms of economy, functionality, aesthetics, sustainability and productivity of the masonry unit. These flaws include efflorescence, low compressive strength, fragmentation, high water absorption and cracking. Different unexpected issues arise for multiple reasons which does not only damage these units but also can become a cause of structure failure. Many efforts have been made around the world to address such issues. The aim of this paper is to review observed flaws in bricks and blocks and to recommend possible sustainable solutions for local construction in Pakistan. The shortcomings of bricks and blocks are discussed in two major contexts societal and environmental concern. A review of various possible improvement techniques investigated by different researcher is presented to overcome the issues. At the end implementable sustainable solutions are recommended for local construction.

Keywords
Additives, Waste materials, Compressive strength, Environment, Admixtures, Society

1. Introduction

Brick and blocks are the most common building material used in Pakistan's masonry construction to render buildings, pavements and other components. The word brick has historically applied to a clay structure, but it is now used to indicate rectangular units made of clay-bearing earth, sand, and lime, or concrete products. Bricks are mostly used in construction of walls of any size, construction of floors, construction of arches and cornices, construction of brick retaining walls, construction of khoa (broken bricks of the required size) to be used as an aggregate in concrete, production of surki (powdered blocks) to be used in lime mortar and lime concrete. A strong square is primarily used in the construction of dividers as a structural material. A solid brick construction unit (CMU) is named here and there. A solid square is one of several solid prefabricated products used in production. The word precast relates to how the squares were originally formed and solidified to be taken to the workplace. Most solid squares have at least one empty cavity, and may throw smoothly or with a plan on their sides. In use, concrete blocks are placed one at a time to form the ideal duration and height of the wall together with fresh concrete mortar.

Recently, the use of renewable agricultural by-products and waste materials as performance-enhancing additives in the brick industry is increasingly gaining ground. During the firing process, the additives mixed in the brick and blocks clay are burning out producing extra energy and decreasing the industrial furnace's total energy requirement. At the beginning, sawdust, wood chips and other wood-based materials were used, but more recently polymers and also renewable agricultural waste materials, like rice-peel or seed-shell are also present as additions in the brick and blocks and tile industry. The ignition of the additives provides additional thermal energy from the inside of the brick product and reduces the furnace energy required.

2. Flaws in bricks and blocks

Sustainability is centre of attraction in the current age, as we consume much amount of natural resources to create materials. The usage of the natural resources is increasing day by day and our resources are limited this puts a pressure sustainable environment arises. Thus, the usage of the waste or by products as an additive in the manufacturing of the brick and block will help to reduce the issues in environmental and societal context and it can also help towards the request of sustainable solution to our problem.

There may be issues with brick and stone masonry, such as splitting and flaking, in homes and buildings. They not only minimize the curb appeal of your house, but they may also be a warning sign of a much bigger problem. In homes and structures, more problems are like efflorescence, strength problems, thermal conductivity, and reuse. Several issues in the use of bricks and blocks as a unit are then met. Excessive moisture enters the bricks and freezes in the winter, Brick erosion can also occur if house or building is old, structural failures in brick walls, efflorescence, concrete blocks water penetration, concrete blocks degradation, leakage and cracking.

Figure 1: Efflorescence's on brick wall

2.1 Issues in bricks and blocks related to societal context

If an historic house or building is constructed with brick, an annual review should be included in the maintenance schedule to determine brick quality. In making this annual effort to review the exterior masonry of house, will identify potential issues that, if not resolved, might turn into major problems. This report gives an outline of block workmanship lodging development, which establishes 62.38% of the all-out fabricated condition of Pakistan. Block and brick work development ranges from regular one-story houses which are normal in rustic zones up to three-story structures (basic in urban zones). Structures of this sort are by and large developed without looking for any conventional designing information. Because of inalienable shortcomings in the basic burden conveying framework and furthermore to the utilization of low-quality development materials, this development type has performed incredibly ineffectively during ongoing tremors in Pakistan. Because of the absence of explicit development rules and the relevant structure license laws to control such development systems, a mind-boggling level of existing just as more current structure stock is presently under an expanded seismic risk (Lodi, 2013).

Since white stains on the substance of block brick work dividers are regularly similar salts that are related with Portland bond, an examination was made to decide if effloresching salts can start in solid reinforcement square. The solvent salts content was resolved for chosen applaud block, solid square, and dirt tile; wick tests were run on the material. Composite dividers were worked of no fluorescent block supported with earth tile or with solid square high stickiness was kept up on the rear of the dividers and the dampness was permitted to relocate to the dry block face. The staining of the face block was thought about and the salts were distinguished (Youthful, 2016).

This report gives an outline of solid square stone work lodging development, which is commonly found in urban regions of Pakistan. Square workmanship covers 3.3% of the all-out assembled condition of Pakistan. Square stone work development is the most widely recognized sort in less created urban regions, where earth is not promptly accessible, and ranges from one-story houses to multistore structures. The development is commonly completed with no specialized information. There are no rules and laws accessible to direct it; in this way, it experiences various shortcomings. This development type is exceptionally defenceless against seismic powers (Lodi, 2012).

2.2 Issues in bricks and blocks related to environmental context

The main environmental impacts of brick making are coal manufacturing, production and combustion, which is the raw material used to burn during fire. Many carbon emissions and pollutants such as SO2 and nitrous oxides are compensated for by using coal in brick production. The work with large amounts of brick kiln toxic elements causes serious health hazards. The brick kilns emit toxic fumes that contain suspended particles matter that is rich in carbon particles and high carbon monoxide and sulphur oxides (SO2) that are harmful to the skin, lungs and mouth.

2.2.1 Burning of fire woods in bricks fired kilns

According to the government rule, brick field owners are charging commercial license, VAT and land development levy, but because of illicit brick production the government is deprived of such revenues. It is prohibited to use firewood in brick field kilns as per the brick burning control law of 1992 and the owners would be punished for violating the law. Raising chimney heights of brick kilns fails to stop grave air pollution bricks were built for building. But the brick kilns of the country churn out the main building ingredients in a way that does more harm than good. Previously, in a bid to reduce harmful carbon emissions from the kilns, the government ordered brick kiln operators to lift their chimneys to 120 ft levels.

2.2.2 Air pollution

Air pollution is characterized as the amount of pollutants that adversely affect humans, plants, animals, or material in the environment. Air pollution is mainly caused by the large-scale burning of coal, diesel, electricity, etc. products to supply energy to domestic industry and transport vehicles. Burning accounts for about a third in air pollution. Air pollution can have direct and indirect consequences. This affects all living beings who breathe or are exposed to polluted air. Particulate things like smaller particles of smoke and dust penetrate deep into the lungs and get deposited there. Sulphur dioxide can frustrate the respiratory system. Carbon monoxide inhalation is also responsible for stomach pain and vomiting.

The value of the business was measured by retail reports in the absence of data on the bricks field. There are around 12,000 brick kilns in Pakistan, according to industry estimates. The kiln industry's global distribution is as limited as it is. The geographical spread of kiln industry is as under.

Table 9 Kiln industry in Pakistan

Locations	No. of kilns	Share (%)
Centre-Upper Punjab	4500	38
Rest of the Punjab	3500	29
NWFP	1000	8
Sindh & Baluchistan	3000	25
Total	12000	100

Among the estimated 12000 brick kilns in the country, some 5500, i.e. 46% are present in the two target regions. Burning is related to high atmospheric carbon emissions which contribute to green- house gases. It occurs when the clay burns at high temperatures and the claystone burns to achieve the high temperatures in the process of making concrete. Here a large carbon footprint. The manufacturing of cement results in high CO_2 output levels. Cement production is the world third-largest anthropogenic (man-made) CO_2 source after transportation and generation of electricity. Cement production contributes about 4-5% of the global CO_2 emissions.

3 Adopted sustainable remedial measures by research community

On observing the present-day interest for blocks, an Endeavor was made to examine the conduct of blocks fabricated utilizing, distinctive waste materials like Rice Husk, Wood Ash, Fly Ash even concrete was utilized to produce blocks. The fundamental point of this undertaking was to analyse the compressive quality of the blocks, so for this reason distinctive level of materials were independently included 4%, 8%, 12% and 16% by weight and afterward the compressive quality of the Bricks was set up, and afterward with the assistance of chart a correlation between compressive quality of blocks, made out of Rice Husk, Wood Ash, earth, Fly Ash and Cement was resolved. Prior to assembling the blocks, various properties of the materials (dirt, wood debris, rice husk, concrete and Fly debris) like sifter examination, explicit gravity was additionally checked (Parashar,2012).

In the examination, a test inquire about was committed to explore the properties of dirt block treated by polyvinyl liquor (PVA). Two techniques for treatment were directed. The main contained treating mud block by submerging the examples in 1%, 2% and 3% of PVA arrangement, while the second included covering the block examples with 6% PVA (for example a high thick arrangement). Standard tests utilized for block examples were applied. Be that as it may, the block examples covered by PVA demonstrated a slight increment in the compressive quality, however a recognizable decline in the water assimilation and flowering (Dhaheer, 2018).

The investigation assessed the impact of an added substance, Al (OH)3, on the singed properties of stubborn earth block. In this examination, four distinction extents of Al (OH)3 to dirt, including 0:100, 25:75, 49:51 and 76:24 were contemplated. The examples of 5x5x5 cm3 were shaped by hand forming, at that point were terminated at 1200, 1300 and 1400 °C. The properties including obstinacy, volume shrinkage, quality, mass thickness, water assimilation, and slaking time were examined. The examples of 25:75 proportion of Al(OH)3 to earth, which was terminated at 1300 co. shown high in unmanageability, low volume shrinkage of 5% and moderately low thickness of 1.69 g/cm3 when contrasted with those of 1400 co. terminated. Additionally, the moderate water adsorption of 15 % and useable compressive quality of 12.4 MPa was likewise watched (Nilpairach,2018).

White stains on the face of brick masonry walls are often the same salts identified with Portland cement, an examination has been undertaken to ascertain if efflorescent salts may derive from concrete backup blocks. For selected clap brick, concrete block, and clay tile, the soluble-salt content was determined; wick tests were conducted on the products. That was achieved by the materials underlying the Effloresce tests, utilizing the ceramic wicks given by the ports and the recommended methods. The materials were ground to pass a 20mesh sieve and 50 gm. of material were exposed to 300 ml. of distilled water in the container in which the Effloresce was inserted (Young, 2016).

Flowering shows up as a for the most part white and thin, foggy salt store on the outside of permeable structure materials. It frequently happens on stone work facades, and relying upon its force, changes the shading impression and energy about the facade overall. The undertaking is done in a few phases following a hereditary methodology dependent on essential investigation of field information. This paper presents the hereditary sorts of flowering experienced during the field study, and an increasingly nitty gritty exchange of one of these sorts, with after effects of a starter blooming test (Brocken, 2004).

Table 10 List of additives used in bricks

Paper No.	Material (Additive)	Percentage Ranges (%)	Paper name
1	Hematite tailings	77-88	Yongliang Chen,(2011)
2	Perlite	24-30	Burak Is-ıkdag,(2007)
3	Aluminium Hydroxide	20-25	Siriphan Nilpairach,(2017)
4	Rice Husk, wood Ash, Fly Ash	4-16	Ashish Kumar Parashar,(2012)
5	Polyvinyl Alcohol	1-6	M.S.Dhaheer ,(2018)
6	Sugarcane bagasse	5-15	Syed M.S. Kazmi,(2016)
7	Marble powder	10-50	Bilgin,(2012)
8	Waste glass	2-5	M.Dondi ,(2009)
9	Fly ash Lime gypsum	20-60	Kumar,(2002)

Table 11 List of additives used in blocks

Paper NO.	Material (Additive)	Percentage Ranges (%)	Paper name
1	Rubberized concrete	5-50	Tung-Chai Ling,(2011)
2	Mud concrete	10	Chameera Udawattha,(2017)

3	Fly ash, Lime Gypsum	20-60	Sunil Kumar,(2002)
4	Glass Material	15-45	Koli Nishikant,(2016)
5	Crumb rubber	10-50	Bashar S,(2012)
6	Compressed earth blocks	5-15	Morel,(2012)

4 Possible solutions for local construction

A massing of unmanaged mechanical or rural strong waste particularly in creating nations has brought about an expanded ecological concern. Reusing of such squanders as a feasible development material has all the earmarks of being reasonable arrangement not exclusively to contamination issue yet in addition an affordable alternative to plan of green structures. Different physic-mechanical and warm properties of the blocks consolidating diverse waste materials are evaluated and proposals are recommended as the result of the examination. The investigated methodology for the structure and advancement of WCB utilizing modern strong waste is helpful to give a potential practical arrangement (Raut, 2011).

Green and centre foundry sand are blended in with earth in extents half and terminated at 850–1050 C to create fired blocks. The examples are physically and mineralogically assessed, the scaling-up dissected, and a streamlining study created. Dirt/green sand blocks terminated at 1050 C have the better physical properties esteems, while the mineralogy isn't fundamentally influenced. The modern quality block can be assessed with research centre preliminaries, and the ideal measure of sand is seen as 35% of green sand and 25% of centre sand (Alonso, 2012).

With the goal of decreasing the negative effects on condition and using the optional asset of tailings, the plausibility of making development blocks by utilizing the hematite tailings from western Hubei territory of China was researched. Other than hematite tailings, the added substances of dirt and fly debris were added to the crude materials to improve the block quality. The ideal conditions were seen as that the hematite tailings content was as high as 84%, framing water content and shaping weight were individually in the scope of 12.5–15% and 20–25 MPa, and the reasonable terminating temperature was gone from 980 to 1030 C for 2 h. The outcomes demonstrated that the fundamental mineral periods of the item were hematite, quartz, anorthite and tridymite, which were mainly liable for the mechanical quality of blocks (Yongliang Chen, 2011).

The examination detailed in this paper is done to think about the achievability of utilizing squashed blocks to substitute the coarse total (rock) in concrete. Two kinds of solid blending are readied. The first is a blend of 1:2:4 without squashed blocks and is utilized as a source of perspective blend. The subsequent one is made of various load of squashed blocks (as a rate from the heaviness of the coarse total). An aggregate of 30 quantities of solid examples are casted with and without squashed blocks and tried under pressure and split strain according to applicable to British standard particulars. Test outcomes showed that utilizing squashed blocks lessens the quality of cement. Additionally, the level of water to concrete (Fadia, 2009).

This examination plans to assess the impact of the waste expansion delivered from two significant harvests: sugarcane and rice in mud blocks fabricating. In this examination, sugarcane bagasse debris (SBA) and rice husk debris (RHA) were gathered locally from a sugar plant and bull's channel furnace, separately. Block examples were produced at a mechanical block furnace plant utilizing different measurements (5%, 10% and 15% by mud weight) of SBA and RHA. Mechanical and strength properties of these blocks were examined. It was seen that earth blocks fusing SBA and RHA displayed lower compressive quality contrasted with that of mud blocks without SBA and RHA. Nonetheless, compressive quality of blocks with 5% of SBA and RHA fulfilled the Pakistan Building Code prerequisites (for example >5 MPa). SBA and RHA (for example 5% by earth weight) won't just calm the natural weight yet in addition result into a progressively manageable and prudent development (Kazmi ,2016).

Table 12 Possible solutions for local construction

Sr.no	**Adoptable solutions**	**Benefits**	**Implemented**	**REFRENCES**
1.	Rice husk, wood ash, earth, fly ash and cement	Compressive quality of blocks and bricks increases and efflorescence decreased	India	Ashish Kumar Parashar, (2012)
2.	Waste glass powder (5% to 40%) by volume	Its effects both Compressive strength, water absorption of bricks	India	Prema Kumar , (2014)
3.	Bio briquette ash (bba), sawdust	Results show that the proposed model building improved the efficiency of indoor temperature control by 23% and, at the same time, reduced the cost by 13%.	India	Vishakha,(2017)
4	Recycled concrete aggregates	75% RCAs satisfied the strength drying shrinkage and freeze-thaw resistance requirements for concrete building blocks	China	Zhanggen, (2018)
5	Jute fibers in stabilized-earth brick (seb)	flexural strength and shear strength are increased	Pakistan	Muhammad Zahid, (2019)
6	Polyvinyl alcohol	Increase in compressive strength and also decrease in effloresces effect	India	Dhaheer, (2018)
7	Sugarcane Bagasse	Reduction in effloresces and water absorption but compressive strength decreased	Pakistan	Syed Kazmi, (2016)

5 Conclusion

In Developing countries like Pakistan where brick and block require ample resources and time, it is important to look for solutions that would be help full in repairing the existing damaged failures and new strategies that would make our structures more durable and serviceable. The arising problems require reintegration which doesn’t only cost material but also energy. To save this Energy and ultimately save the Environment It is the need of the hour to make more durable and serviceable structures. Brick and block work issues, for example, breaking and chipping, can be available in homes and structures. In addition to the fact that they decrease your home's check claim, however they could likewise be an admonition indication of a lot bigger issue. More issues in homes and structures resemble efflorescence’s, strength, sustainability, social and environmental. Following are some problems of bricks out of them we will only solve few and will try to solve all of them by testing them and using different strategies.

6 Acknowledgment

Foremost, I would like to express my sincere gratitude to all the researchers my group members Engr.Dr. Majid Ali and Engr.Faiza Khalid for the continuous support of my conference paper and for their patience, motivation, enthusiasm, and immense knowledge. Their guidance helped me in all the time of research and writing of this paper.

7 References

Parashar, A. K., & Parashar, R. (2012). Comparative study of compressive strength of bricks made with various materials to clay bricks. International Journal of Scientific and Research Publications, 2(7), 1-4.

Mohammed, B. S., Hossain, K. M. A., Swee, J. T. E., Wong, G., & Abdullahi, M. (2012). Properties of crumb rubber hollow concrete block. Journal of Cleaner Production, 23(1), 57-67.

Bilgin, N., Yeprem, H. A., Arslan, S. Ö. N. M. E. Z., Bilgin, A., Günay, E., & Marşoglu, M. (2012). Use of waste marble powder in brick industry. Construction and Building Materials, 29, 449-457

Topçu, İ. B., & Işıkdağ, B. (2007). Manufacture of high heat conductivity resistant clay bricks containing perlite. Building and Environment, 42(10), 3540-3546.

Udawattha, C., & Halwatura, R. (2017). Character of lime as an alternative stabilizer to improve the long term strength of mud concrete block. In 8th International Conference on Structural Engineering and Construction Management, ICSECM2017.

S Kallak, F. (2009). Use of crushed bricks as coarse aggregate in concrete. Tikrit Journal of Engineering Sciences, 16(3), 64-69.

Nishikant, K., Nachiket, A., Avadhut, I., & Sangar, A. (2016). Manufacturing of concrete paving block by using waste glass material. International Journal of Scientific and Research Publications, 6(6), 61-77.

Lodi, S. H., Abdul, J. S., & Adam, A. (2012). Concrete-block masonry construction in Pakistan. World Housing Encyclopedia by Earthquake Engineering Research Institute and International Association of Earthquake Engineering.

Dondi, M., Guarini, G., Raimondo, M., & Zanelli, C. (2009). Recycling PC and TV waste glass in clay bricks and roof tiles. Waste management, 29(6), 1945-1951.

Dhaheer, M. A., Ammash, H. K., & Albdiry, M. T. (2018). Improving the Properties of Clay Brick Using Polyvinyl Alcohol (PVA). International Journal of Engineering & Technology, 7(4.20), 568-571.

Alonso-Santurde, R., Coz, A., Viguri, J. R., & Andrés, A. (2012). Recycling of foundry by-products in the ceramic industry: Green and core sand in clay bricks. Construction and Building Materials, 27(1), 97-106.

Raut, S. P., Ralegaonkar, R. V., & Mandavgane, S. A. (2011). Development of sustainable construction material using industrial and agricultural solid waste: A review of waste-create bricks. Construction and building materials, 25(10), 4037-4042.

Nilpairach, S., Watchaikun, A., Panyawatcharakom, K., Wasanapiarnpong, T., & Jiraborvornpongsa, N. (2018). Effect of Aluminum Hydroxide Addition on Properties of Fired Refractory Clay Brick. In Key Engineering Materials (Vol. 766, pp. 300-304). Trans Tech Publications.

Kumar, S. (2002). A perspective study on fly ash–lime–gypsum bricks and hollow blocks for low cost housing development. Construction and Building Materials, 16(8), 519-525.

Kazmi, S. M., Abbas, S., Saleem, M. A., Munir, M. J., & Khitab, A. (2016). Manufacturing of sustainable clay bricks: Utilization of waste sugarcane bagasse and rice husk ashes. Construction and building materials, 120, 29-41.

Ling, T. C. (2011). Prediction of density and compressive strength for rubberized concrete blocks. Construction and Building Materials, 25(11), 4303-4306.

Chen, Y., Zhang, Y., Chen, T., Zhao, Y., & Bao, S. (2011). Preparation of eco-friendly construction bricks from hematite tailings. Construction and Building Materials, 25(4), 2107-2111.

Oti, J. E., & Kinuthia, J. M. (2012). Stabilised unfired clay bricks for environmental and sustainable use. Applied Clay Science, 58, 52-59.

Chen, Y., Zhang, Y., Chen, T., Zhao, Y., & Bao, S. (2011). Preparation of eco-friendly construction bricks from hematite tailings. Construction and Building Materials, 25(4), 2107-2111.

Sakhare, V. V., & Ralegaonkar, R. V. (2017). Development and investigation of cellular light weight bio-briquette ash bricks. Clean Technologies and Environmental Policy, 19(1), 235-242.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March13-14, 2020, Karachi, Pakistan.

A Review on Different Influential Factors and Techniques for Evaluating Bonding between Asphalt Overlay and Concrete Pavement

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Abstract
Interlayer bonding state has a significant effect on the performance and behavior of the road, in a layered system of asphalt overlay on concrete pavement. However, there still remains the lack of knowledge about significant factors and techniques for testing this phenomenon; because several factors affect the bonding performance between asphalt and concrete layers; and it involves observation through various destructive and non-destructive test analyses. Hence, there is a need to review advance strategies for the evaluation of such bonding. The aim of this paper is to charter the different techniques that can be used to evaluate the inter-facial bonding between asphalt overlay and concrete pavements. First, the mechanism of asphalt overlay on concrete pavement is explained in detail and then different types of observed de-bonding mechanisms are discussed. This review also highlights the effectiveness of different non-destructive and destructive tests for the evaluation of bonding between the two layers. Every technique that can be used for evaluation has been considered under specific condition. However, each technique has its own limitations and results regarding the factors affecting interlayer bonding. Their pros and cons are also described. Recommendations are made for evaluating the bonding with reliable methods for developing countries so as to address the issue of de-bonding and its relative impacts on layers.

Keywords
Bituminous Pavement, Asphalt Layer, Tack Coat, Bond Strength, Shear Test

1. Introduction

With the evolution of advance strategies and applications, the performance of asphalt overlay and concrete pavement has enhanced. The analysis of different failure outcomes and their possible causes has enhanced the type of material used and construction practices. The selection of material and its quanitity used, combining with traffic load and induce various failures in the pavement. Hence the bond between asplaht overlay and concrete below serves as the most significant factor in the occurance of pavement failures that include slippage and other cracking, and thus involves the major research work all over the world to overcome these defects. [Reference 1-4]

In order to improve the bond between these two layers, bond of bituminous emulsions consisting of carbohydrate binder disolved in water with charged emulsion is used. This bond is commonly known as tack coat. The emulsifier enables the dispersion of binders and water, thus preventing the to form anglomerates and allows the adherence and resistence by the aggregates. Thus when the binder particles and water seperates and the binder particles comibined again, they form a bituminous layer. These emulsifiers come into different catergories that are slow, medium and fast breaking emulsifier. Hence the fast breaking emulsifiers are applied for tack coats.
It has been standardized on site that the emulsions being used as tack coats must have more than 200g/m^2 of binder and more than 250g/m^2 in case the top layers is discontious and of hot mix asphalt ir semi-

dense asphalt. However, this standard becomes invalid due to some factors that include types of aggregate and asphalt being used. Moreover, the breakage time of emulsion is not available aswell.

2. Inter-layer Bond Test

Different type of techniques and tests are used to analyze the quallity of tack coats for pavement layers that include shear, tensile and torque tests.

2.1 Tensile Test

There are different reserches where the tensile strength have been considerd. Major tests apparatus and researchers were Litzka et al. [2] who used Schenck-Trebel Test and analyzed different combinations of asphalt test specimens. Furthermore, Intecasa developed the ENDACMA tensile test, a Schenck-Trebe l-based Test device, but the results obtained were not accurate. In 1995 Tschegg et al. [5] proposed Pull-Off Test, in which the top layer was attached to a steel plate to create a vertical load on it and the bottom layer was fixed to the pavement and it produced the layer de-bonding. Later, Tschegg [6] in 1997 proposed the Wedge-Splitting Test. In this test, the wedge applied load on the bonded surface to separate two layers. Instro Tek proposed the ATacker device [7], in 2003, to analyze the longitudinal strength and analysed the various surfaces. This phenomenon included placing the test specimen on a steel plate after applying a tack quote on it and then applying tensile load on it thus resulting in the seperation of layers. Other similar research based tests included UTEP Pull-off Device [8] and lousina Tack Coat Quality Tester (LTCQT) [9].

2.2 Torque Test

The British Bond Agreement first acrediated the first torque test as Torque Bond Test in 1998 [10]. This test applied torque forse on the test specimen and top layer which resulted in debonding of layers. Furthermore, with this basic approach Collop et al. [11] implemented an automatic Torque Bond Test. Hence, here torque was applied instead of vertical tensile load.

2.3 Shear Tests

Uzan et al. [12] in 1978 produced relation between asphalt layers and its bondings. The research included a test specimen with two layers. Vertical load was applied to the above layer and horizontal load to the mould above. Furthermore, similar research work with modifictions were considereed by ASTRA (Ancona Shear Testing Research and Analysis) [13,14] and SuperPave Shear Test [15].

Another strenuous work was done by Leutner [16] in which test specimen was placed inside the shear cast and the load was applied on the top layer while the bottom layer was restricted to create displacement between layers. This practice was included in German Standard [17]. Later The LPDS Tester [18], the FDOT (Florida Department of Transportation) Bond Strength Device [20], the LCB (Laboratory of Civil engineering of Barcelona) Shear Test [19], the ALDOT–NCAT Bond Strength Device [21] and the Louisiana Interlayer Shear Strength Tester [22], were also developed later based upon Leutner test apparatus. The LPDS was added in Swiss Standard SN 670 461 [23], and the LCB Shear Test was included in the NLT-328/08 Standard to analyze the shear strength between the layers of the pavement. Later Raab et al. [25] analyzed that in some cases the strength of the layers and stiffness value decreases if the gap of width is increased.

In 1996, Millien et al. [26] demonstrated Double Shear Test, in which three asphalt layers with tack coat on interfaces was analyzed and a specific load was applied on the intermediate layer to produce the shear on the joints faces. Later, Romanoschi and Metcalf [27] in 1999. The Normal load and the Direct Shear Load , in which the normal load is applied near to the bonded surface of two layers. Moreover, varying temperatures were also considered and Shear Fatigue Test, in which the test specimen was provided a lateral displacement of faces and broken, at specific inclination angle.

With an experimental research of various inter-layer bond verification test, it can be determined that the most applicable and commonly used method for determining the inter-layer bond behavior is shear test. This test replicates the real scenarios of slippage and cracking between layers. Thus, an accurate non-destructive test method is to be developed to increase the accuracy of usage of tack coat on site. Hence, constituting different other factors, the most efficient test method would be proposed.

3. Factors Affecting Inter-layer Bond Strength

Uzan et al. [12] analyzed bond between layers in 1978 and since then many researcher have proceeded to determine the driving factors for the effectiveness of the tack coats. Hence, the major factors considered were the type of binder, quantity of binder, bituminous mixture type, inter-facial surface characteristics, load and temperature during its application. Thus, different suitable proposals were obtained.

3.1 Tack Coat Characteristics

There are certain characteristics of the tack coats that implies and effect on inter-layer bond strength of pavements. Hence, each characteristic has been keenly observed and noted.

3.1.1 Binder dosage

Many researchers have experimented the strength of different test specimens with and without binder between inter-facial layers to examine the effect of use of binders. Romanoschi and Metcalf [28] used modulus from curve slope, coefficient of friction and maximum shear strength. Hence, only modulus of curve slope and shear strength were affected without the application of tack coat. Diakhate et al. [29] observed that the fatigue performance decrease without the application of tack coat.

It is also important to consider the results of other binder dosages without tack coats. Hence, Molenaar et al. [30] and Piber et al. [33] held their research techniques under no binder conditions. In some conditions better results for the inter-layer bond could be obtained without the usage of tack coats [34]. This could be obtained with high temperature of bituminous mixtures as the contact area of layer surface could be bonded with the layer of bitumen. On the other hand, the other results also conclude that the excess of tack coats can also generate slippage between bonded layers [32].

Similarly, Sholar et al. [32,20], Zamora-Bar raza et al. [35] and Raposeir as et al. [36] studied different tack coat usages.

Therefore, a study with various emulsion types would be helpful, because conventional, modified emulsions and asphalt binders have different performance, water ratio and usage characteristics.

3.1.2. Type of binder

The binder type has a significant impact on bond characteristics in terms of strength. West et al. [21], Liu and Hao [37] and Du [38] established that the type of tack coat is more important than its quantity used. They analyzed that the heat-adhesive emulsions are more preferable than conventional emulsions, which gives better performance when applied between the two layers and thus improves the strength in terms of bearing traffic loads.

Moreover, numerous surveys of state department s of transportation , Asphalt Institutes and contractors were done [39]. The results concluded that the asphalt emulsion is an ideal element to asphalt cement and slow-setting is chosen over fast-setting emulsion. Hence, this emulsion type is not widely used.

Different researchers including Bae et al [42], Leng et al. [43], Du [38], Muhammad et al. [41], Collop et al. [11] and Zamora-Barraz a et al. [35] have produced different observational analysis with different binder dosages and type. In some analysis three types of emulsified tack coats (trackless, CRS-1 and SS-1) at varying dosages (0.14, 0.28, 0.70), and the maximum inter-surface bond strength was obtained from trackless tack coat and the lowest strength was of CRS-1 [41]. Zamora-Barraz a et al. [35] observed that the emulsions, most appropriate as heat adhesives are with the dosage from 300 to 400 g/m^2 of residual bitumen.

Moreover, the other researchers have conducted other analysis for better results for various other types of binders. Liu and Hao [37] concluded better fatigue behavior of asphalt binders. Better rutting strength for PG 64-22 and SS-1hP was obtained by Leng et al. [43]. Highest bond strength for catonic emulsion was obtained by Collop et al. [11]. Furthermore, an increase in the rheology of the binder results in enhanced bond strength [42]. Fatigue damages are observed more in many of tack coats materials in thin structures than thick structures, which tend to have better resilience against such failures [44].

With new advancements in research-work and comparative analysis of different binders, it is required to carry out efficient analysis to compare results. As per experimental observations it has been observed that heat-adhesives are better in performance than conventional binders. Hence, the breaking time of the binders has a significant role so this element needs to be considered.

3.1.3 Breaking time

The breaking time of the emulsion was also considered by Tashman et al. [45] , Chen et al. [46] and Deysarkar [8] to observe the impacts on the relative bond strength between two layers. Furthermore, in order to achieve appropriate results from tack coat, a curing time of 40-50 minutes per gram of asphalt emulsion was established [36].

3.2 Surface Characteristics

Immense importance is attached to the surface conditions. Results radiating under different surface characteristics are noted, carefully.

3.2.1 Type of material

The material used also plays a significant role in the strength between the corresponding layers for the construction of the pavements. Different materials were observed by Caltabiano et al. [1], Chen et al. [47], Raab et al. [18] and Utzan et al. [12]. They observed that the flaws in pavements that majorly include slippage crack were connected to the poor quality of tack coat, asphalt content and aging rate [47]
West et al. [21], Mohammad et al. [44] and Raab et al. [48,25] considered the type of mixtures to obtained more accurate data and they observed that the content of void in bituminous mixtures has an influential role on the behavioral characteristics of the bond strength of two layers. Hence from these studies it has been deduced that the types of bituminous mixtures have driving effect on final bond strength.

3.2.2. Surface state

Surface states are also observed to analyze the the surface characteristics. Sholar et al. [20], Raab and Partl [32], Leng et al. [43] and Mohammad et al. [39] have constituted various surface conditions, that includes moisture and dust, in their analysis. They summed-up to an observation that the pavement which has no tack coat used has least performance and adherence in moisture [32]. Even the bonding also reduces due to the presence of moisture on the surface of the layer [20].

Different and varying results were also obtained by Mohammad et al. [39]. Same results have been obtained with wet and dry conditions, whereas dusty and clean conditions have different results. There has been observed an impact of Portland Cement Concrete (PCC) cleaning methods on Hot Mix Asphalt (HMA) and Plain Cement Concrete (PCC) inter-facial bonding in [43].

3.3. Temperature

Another important aspect having salient impact on inter-facial bonding is the temperature and its relative implications, as since it directly affect the behavior of the binders. And hence, varying temperature could alter the characteristics of the bitumen.

With detailed observations, Romanoschi and Metcalf [27], Partl and Raab [54], West et al. [21], Recasens et al. [50,51], Piber et al. [33], Diakhate et al. [29, Chen and Huang [46], Canestrar i and Santagata [49],Du [38] and Mohammad et al. [15] found that the impacts of temperature on bond strength is yet another salient factor. The temperature has great impacts on the strength [38] and the coefficient of friction, curve slope and maximum slope are greatly altered by change in temperature [28]. The inter-layers surfaces have significant influence from the high temperature [46]. And also bitumen softening point temperature [9] has been developed to analyze the tack coat quality, as well.

West et al. [21], Bae et al. [42], Collop et al. [11] and Du [38] also found in their research that the bond strength decreased when temperature increased. This is due to the emulsion characterist ics, because when temperature increased, the emulsion reached the softening point and became more liquid, with adecrease of bond strength. Moreove r, longer lifetime under fatigue and greater sen- sitivity to shear stress levels were found for lower temperatures [11].

It has also been observed by Deysarkar [8] that the bond strength increases under high temperature, as the water evaporates faster from the tack coat and thus catalyzing the emulsion break, allowing it to form a bituminous layer between two surface.

Fine results were obtained for heat-adhesive emulsions, by Bae et al. [42], which indicated that the non-modified emulsions are suitable to be used under high temperature and modified emulsions to be used under low temperatures, based upon their performance.

Thus, it is indicated that the variations in temperatures can navigate the performance of various bituminous emulsion and their relative performances. A decrease in temperature can increase bond strength but this also depends upon the area where it is being considered. The areas where the temperature is relatively less, conventional emulsions should be applied for attaining inter-layer bond strength. And the areas where the temperature is relatively higher than non-modified heat-adhesive emulsions are suggested to be used. However, modified heat-adhesive emulsions could also be used or favorable conditions.

4. Conclusions

After an adequate analysis of various studies and observational research work, it has been deduced that the most appropriate method to check the inter-layer bond strength, of pavement layers, are the shear tests. The shear test sufficiently replicates the real behavior of de-bonding and slippage of layers under various conditions. The LPDS Tester and LCB shear Test are the devices used for the shear tests, based upon Leutner method.
The tack coat is an important element, which is used to increase the bond strength. However, insufficient usage of tack coat can decrease the bond strength and likewise, an excess can induce slippage between layers of the pavement.

Moreover, the type of binder has less impact upon the final bond strength. The use of heat-adhesives emulsions are also suggested to be used. However, conventional anionic emulsions, asphalt binders and conventional cationic emulsions

Temperature also plays a significant role in the behavior of bond strength. Any variation in temperature could majorly alter the characteristics of the asphalt binder and emulsions. Conventional emulsions are suggested to be used in low temperatures, whereas the heat adhesive emulsions are most appropriate to be used in high temperatures.

5. References

[1] Caltabiano MA, Brunton JM. Reflection cracking in asphalt overlays, asphalt paving technology. United States: Assoc of Asphalt Paving Technologists, St. Paul MN; 1991. p. 310.

[2] Litzka JH, Pass F, Zirkler E. Experiences with thin bituminous layers in Austria. Transp Res Rec 1994;1454:19–22.

[3] Raab C, Partl MN. Investigation into a long-term interlayer bonding of asphalt pavements. Baltic J Road Bridge Eng 2008;3(2).

[4] Raab C, Partl MN. Interlayer bonding of binder, base and subbase layers of asphalt pavements: long-term performance. Constr Build Mater 2009;23(8):2926–31.

[5] Tschegg EK, Kroyer G, Tan D, Stanzl-Tschegg SE, Litzka JH. Investigation of bonding between asphalt layers in road construction. J Trans Eng 1995;121(4):309–16.

[6] Tschegg EK. An efficient fracture test method for bituminous materials and layer bonds. In: Re ´ union internationale des laboratoires et experts des materiaux, syste`m es de construction et ouvrages (RILEM) [international union of laboratories and experts in construction materials, systems, and structures]. Liege (Belgium): RILEM; 1997. p. 405.

[7] InstroTek Inc. ATACKER™ A Tack Coat Testing Device. USA; 2003.

[8] Deysarkar I. Test set-up to determine quality of tack coat. University of Texas at El Paso; 2004.

[9] Mohammad LN, Bae A, Elseifi MA, Button J, Scherocman JA. Evaluation of bond strength of tack coat materials in field: development of pull-off test device and methodology. Transp Res Rec 2009;2126(1):1–11.

[10] British Board Agreement. Guideline document for the assessment and certification of thin surfacing systems for highways; 1998.

[11] Collop AC, Sutanto MH, Airey GD, Elliott RC. Development of an automatic torque test to measure the shear bond strength between asphalt. Constr Build Mater 2011;25(2):623–9.

[12] Uzan J, Livneh M, Eshed Y. Investigation of adhesion properties between asphaltic-concrete layers. Asphalt Paving Technol 1978;47:495–521.

[13] Santagata E, Canestrari F. Tensile and shear tests of interfaces in asphalt mixes: a new perspective on their failure criteria. In: Proceedings of the 2nd international symposium on highway surfacing; 1994.

[14] Santagata E, Canestrari F, Santagata FA. Laboratory shear testing of tack coat emulsion. In: Proceedings of the 1st congress on emulsion; 1993.

[15] Mohammad LN, Raqib MA, Huang B. Influence of asphalt tack coat materials on interface shear strength; 2002.

[16] Leutner R. Investigation of the adhesion of bituminous pavements [untersuchungen des schichtenverbunds beim bituminösen oberbau]. Bitumen 1979;3:84–91.

[17] Forschungsgesellschaft für Straben- und Verkehrswesen. Testing of layered composite according to Leutner. Sampling and analysis [prüfung des schichten-verbundes nach Leutner. Probenahme und durchführung der prüfung]. 1st ed. Berlin (Germany): Society for Research and Road Transport, German Standard; 1998.

[18] Raab C, Partl MN. Methods to determine the bond of asphalt pavements [methoden zur beurteilung des schichtenverbunds von asphaltbelägen]. ASTRA-project FA 12/94; 1999.

[19] Recasens RM, Jiménez FP, González JMB. Partl MN. Evaluation of the effect of tack coats. LCB shear test; 2003. p. 550–56.

[20] Sholar GA, Page GC, Musselman JA, Upshaw PB, Moseley HL. Preliminary investigation of a test method to evaluate bond strength of bituminous tack coats. Asphalt Paving Technol 2004;2004:771.

[21] West RC, Zhang J, Moore J. Evaluation of bond strength between pavement layers. National Center for Asphalt Technology, Auburn University, Alabama; 2005.

[22] Mohammad LN, Bae A, Elseifi MA, Button J, Patel N. Effects of pavement surface type and sample preparation method on tack coat interface shear strength. Transp Res Rec 2010;2180(1):93–101.

[23] Schweizer Norm SN 670 461 2000. Determination of the adhesion between asphalt layers (accordingto Leutner)[bestimmug des schichtenverbunds (nach Leutner)].5th ed. Switzerland: Swiss Standard, Zürich.

[24] NLT-328/08. Evaluation of the adhesion between asphalt layers, through shear test [evaluacuón de la adherencia entre capas de firme,mediante ensayo de corte]. 1ª ed. Madrid: CEDEX Spanish Standard; 2008.

[25] Raab C, Partl MN, Abd El Halim AO. Effect of gap width on interlayer shear bond results. Int JPavement Res Technol 2010;3(2):79–85.

[26] Millien A, Petit C, Rosier J. Shear behavior of the tack coats on pavements [comportement au cisaillement des couches d'accrochage dans les chausse ́ es]. Rapport Int 1996.

[27] Romanoschi SA, Metcalf JB. Characterization of pavement layer interfaces; 1999.

[28] Romanoschi SA, Metcalf JB. Characterization of asphalt concrete layer interfaces; 2001.

[29] Diakhate M, Millien A, Petit C, Phelipot-Mardele A, Pouteau B. Experimental investigation of tack coat fatigue performance: towards an improved lifetime assessment of pavement structure interfaces. Constr Build Mater 2011;25(2):1123–33.

[30] Molenaar AAA, Heerkens JCP, Verhoeven JHM. Effect of stress absorbing membrane interlayers. In: Proceeding of the association of asphalt paving technologists, vol. 55; 1986. [31] Collop AC, Thom NH, Sangiorgi C. Assessment of bond condition using the Leutner shear test. Proc Inst Civil Eng: Trans 2003;156(4):211–7.

[32] Raab C, Partl MN. Effect of tack coats on interlayer shear bond of pavements. In: Proceedings, 8th conference on asphalt pavements for Southern Africa; 2004.

[33] Piber H, Canestrari F, Ferrotti G, Lu X, Millien A, Partl MN, et al. RILEM interlaboratory test on interlayer bonding of asphalt pavements. In: 7th international RILEM symposium on advanced testing and characterisation of bituminous materials. Rhodes (Greece): RILEM; 2009. p. 1181–9.

[34] Collop AC, Sutanto MH, Airey GD, Elliott RC. Shear bond strength between asphalt layers for laboratory prepared samples and fieldcores. Constr Build Mater 2009;23(6):2251–8.

[35] Zamora-Barraza D, Calzada-PerézM, Castro-Fresno D, Vega-Zamanillo A. New procedure for measuring adherence between ageosynthetic material and a bituminous mixture. Geotext Geomembranes 2010.

[36] Raposeiras AC, Vega-Zamanillo A, Calzada-PérezMA, Castro-Fresno D. Influence of surface macro-texture and binder dosage on the adhesion between bituminous pavement layers. Constr Build Mater 2012;28(1):187–92.

[37] Liu L, Hao PW. Investigation of interface shear fatigue performance and evaluating index between asphalt layers. In: International workshop on energy and environment in the development of sustainable asphalt pavements, proceedings. China: Changan Univ.; 2010.

[38] Du J. Evaluation of shear strength on pavement layers by use tack materials. Adv Mater Res 2011;255–260(1):3176–9.

[39] Mohammad LN, Saadeh S, Qi Y, Button J, Scherocman J. Worldwide state of practice on the use of tack coats: asurvey. In: Asphalt paving technology: association of asphalt paving technologists – proceedings of the technical sessions. Texas, USA: Texas Transportation Institute, Texas Aand MUniversity System, College Station; 2008. p. 1–34.

[40] Raab C, Partl MN. Influenceof tack coats on the adhesion properties of asphalt pavements. In: Efficient transportation and pavement systems: characterization, mechanisms, simulation, and modeling –proceedings of the 4th international gulf conference on roads. Duebendorf (Switzerland): Department of Road Engineering and Sealing Components, Empa. p. 517–27.

[41] Mohammad LN, Bae A, ElseifiMA, Button J. Scherocman JA. Interface shear strength characteristics of emulsifiedtack coats. In: Asphalt paving technology: association of asphalt paving technologists –proceedings of the technical sessions. Texas (USA):Texas Transportation Institute, Texas A. and M. University; 2009. p. 249.

[42] Bae A, Mohammad LN, ElseifiMA, Button J, Patel N. Effects of temperature on interface shear strength of emulsifiedtack coats and its relationship to rheological properties. Transp Res Rec 2010;2180(1):102–9.
[43] Leng Z, Al-Qadi IL, Carpenter SH, Ozer H. Interface bonding between hot-mix asphalt and various Portland cement concrete surfaces: assessment of accelerated pavement testing and measurement of interface strain. Transp Res Rec 2009;2127(1):20–8.
[44] Mohammad LN, Hassan M, Patel N. Effects of shear bond characteristics of tack coats on pavement performance at the interface. Transp Res Rec 2011;2209(1):1–8.
[45] Tashman L, Nam K, Papagiannakis T, Willoughby K, Pierce L, Baker T. Evaluation of construction practices that influence the bond strength at the interface between pavement layers. JPerform Constr Facil 2008;22(3):154–61.
[46] Chen JS, Huang CC. Effect of surface characteristics on bonding properties of bituminous tack coat. Transp Res Rec 2010;2180(1):142–9.
[47] Chen DH. Slippage failure of anew hot-mix asphalt overlay. JPerform Constr Facil 2011;24(3):258–64.
[48] Raab C, Abd El Halim AO, Partl MN. Interlayer bond testing using amodel material. Constr Build Mater 2012;26(1):190–9.
[49] Canestrari F, Santagata E. Temperature effects on the shear behaviour of tack coat emulsions used in flexiblepavements. Int JPavement Eng 2005;6(1):39–46.
[50] Recasens RM, MartínezA, JiménezFP. Evaluation of effect of heat-adhesive emulsions for tack coats with shear test: from the road research laboratory of Barcelona; 2006.
[51] Recasens RM, Martínez A, Jiménez FP. Assessing heat-adhesive emulsions for tack coats. Proc Inst Civil Eng: Trans 2005;158(1):45–51.
[52] Santagata FA, Ferrotti G, Partl MN, Canestrari F. Statistical investigation of two different interlayer shear test methods. Mater Struct/Mater Constr 2009;42(6):705–14.
[53] UNE-EN 13108–1. Bituminous mixtures. Material specifications. Part 1: Asphalt concrete. [Mezclas bituminosas. Especificaciones de materiales. Parte 1: Hormigón bituminoso]. European Standard ed.; 2008.
[54] Partl MN, Raab C. Shear adhesion between top layers of fresh asphalt pavements in Switzerland. In: 7th Conference on asphalt pavements for Southern Africa; 1999.
[55] Khweir K, Fordyce D. Influence of layer bonding on the prediction of pavement life. Proc Inst Civil Eng: Trans 2003;156(2):73–83.
[56] Diakhate M, Millien A, Phelipot-Mardele A, Petit C. Fatigue behaviour of tack coats in the pavement structures under shear [comportement à la fatigue en cisaillement des couches d'accrochage dans les structures de chaussée]. In: 22 ème rencontres universitaires de génie civil; 2004. p. 3–4.
[57] Muench ST, Moomaw T. De-bonding of hot mix asphalt pavements in Washington State: an initial investigation. 1st ed. Washington: Washington State Dept. of Transportation, Office of Research & Library Services; 2008.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Effect of Fly Ash and Polypropylene Fiber on Compressive Strength of Concrete at Normal as Well as Elevated Temperatures

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Abstract
Fly ash has fire resisting properties and improves the mechanical properties of concrete, but it cannot make the structures serviceable up to 28 days due to low early strength. For the improvement of strength at early ages, 4 % aqueous sodium silicate is added in concrete in this paper. Polypropylene (PP) fibers (0.5 % by volume of concrete) with fly ash (40 % replacement of cement) were utilized to check the compressive strength both at normal and elevated temperature. No spalling occurred with testing temperature 100-650°C at elevated temperature by using PP fiber. By using activator with fly ash in concrete, the compressive strength of activated fly ash concrete (CAF) was 52.15 and 29.80 % greater than inactivated fly ash concrete (CIF) at 07 and 28 days, respectively. Use of fly ash enhanced concrete strengths elevated temperatures especially up to 200 °C. For elevated temperatures up to 650°C, the compressive strength of PP fiber reinforced concrete with activated and inactivated fly ash was higher than that of the control concrete.

Keywords
Compressive strength, spalling, polypropylene fiber, elevated temperature, activated and inactivated fly ash concrete.

1. Introduction

Rapid growth in civilization has compelled research community to develop strong, durable and cost-efficient materials to provide safe shelter to humanity. There is need of innovative materials which should be able to withstand high impact of natural disasters such as extreme temperatures etc. and accidental damages such as fires to provide safety to residents and prevention of high damage structures to save economy. The construction industry of the whole world is using cement.

Fly ash, which is a byproduct of combustion of coal in the generation of electricity, is used now a day in concrete widely. It produces an environment friendly concrete and when used in concrete as replacement of cement in an optimum percentage, the mechanical properties of concrete could also be enhanced (Hefni, El Zaher, and Wahab 2018). Although concrete behaves good against fire resistance, but at the higher temperature the physical and chemical changes occur in concrete structure which changes its mechanical properties. As compared to normal concrete, the pressure is built up in the high strength concrete due to very low permeability in it (Biolzi, Cattaneo, and Rosati 2008). The spalling and mechanical properties change at higher temperature basically depends upon the heating conditions, type of aggregate, permeability and moisture content. Basically, at higher temperature the rapid removal of hydroxyl ions from the compounds like calcium hydroxide occurs and calcium-silicate-hydrate (C-S-H) gel starts decomposition (Janotka and Mojumdar 2005).

In concrete construction, fibers are used as it improves mechanical properties such as strength, ductility, flexure, tensile, energy of fracture, shrinkage and resistance to spalling at higher temperature (Janotka and

Mojumdar 2005). Polypropylene fibers believed to enhance the thermal stability as it dissolves and melt at 160 – 170 °C and creates additional channel resulting in more pores in concrete structure. This enables the concrete not to crack and spall but can also decrease the mechanical properties of concrete (Afroughsabet and Ozbakkaloglu 2015) (Zhang and Li 2013). It has also been shown that polypropylene fibers in concrete at normal temperatures increases the durability of composite concrete which contain the fly ash and silica fumes (Khaliq and Kodur 2011). The 20% replacement of cement with fly ash lowers the early ages but improves compressive strength at later ages (Alsalman, Dang, and Hale 2017). At higher temperature, the strength in compression of plain cement concrete (PCC) as compared to the fiber reinforced concrete is less. Polypropylene fibers when heated to higher temperature (400-600°C) results in severe loss in compressive strength due to decomposition of calcium hydroxide. Further, when temperature exceeds up to 800 °C, loss in compressive strength occurs rapidly due to deterioration of concrete(Khaliq and Kodur 2011). At higher temperature the bond becomes very weak between aggregate and the paste of cement because of loss of water and expansion of aggregates (Uysal 2012). The addition of activated fly ash with different fibers helps to maximize the advantages of fly ash in concrete. With the addition of polypropylene fibers with fly ash in concrete, it leads to increased strength and greater fire resistance (Hefni, El Zaher, and Wahab 2018).

When exposed to the elevated temperatures, the foremost difference between high strength concrete (HSC) and normal strength concrete (NSC) is because of the tense spalling. The use of PP fiber is suggested to eradicate the danger of explosive spalling in HSC at high temperature. PP fibers have the advantage of reducing the plastic shrinkage cracking, in addition to that PP fibers increase the ductility, hardness, along with that it enhances the impact resistance of concrete. Because of these benefits of PP fibers, they are used in floor systems, pavements, slabs, precast pile members and canals. When normal HSC is exposed to elevated temperature then explosive spalling occurs and there are more chances of severe spalling as the temperature goes on increasing. Spalling is specifically an important phenomenon in dense concrete (micro structured) when exposed to fire at elevating temperatures. Researchers have suggested the addition of polypropylene fibers in concrete to eradicate the fire tempted spalling in HSC (Ali et al. 2004) PP fibers melt at a lower temperature of 167-170 °C and produce micro channels or macro channels in the concrete which assist in the dispersion of higher vapor pressure (Kalifa, Chene, and Galle 2001).

This research is carried out to explore the effect of polypropylene fibers with fly ash with and without activation on concrete mechanical properties at elevated as well at normal temperature. The research study also aims to explore the effect of polypropylene fibers with fly ash with and without activation on mechanical properties of concrete at elevated temperatures. The importance of the proposed paper is based on the use of fly ash through chemical activation, which will help to maximize the use of fly ash in a wide range of applications in the concrete industry. Replacing cement in concrete can reduce environmental pollution and preserve natural resources. The selection of polypropylene fibers containing fly ash in concrete will have environmental, economic and structural advantages, such as increased strength and fire resistance of the concrete structures.

2. Experimental Program

In this paper a total of 198 concrete specimens consisting of cubes were tested for compressive strength. This part of the paper consists of the methodology of research; materials selection, material properties, and casting, testing and heating of samples.

2.1 Materials

2.1.1. Cement

In experimentation, the Ordinary Portland Cement (OPC, Type-I) by Fauji Cement Company is used as per ASTM C 150/C 150 M-18 standard.

2.1.2. Aggregates

Natural Lawrencepur /Harrow /Well graded sand which has fineness modulus of 2.75 is used as fine aggregate in concrete. Margalla crush aggregate was used in this research according to the Standard ASTM C33/ C33 M-18. Two different sizes were used in concrete. 70 % of aggregate passes through 37.5 mm sieve while the other 30 % aggregate passes through 25 mm sieve size. The fineness modulus for coarse aggregate is 3.35. The passing percentages of fine, coarse, and mixture aggregates and the grading curve is shown in the Figure 1.

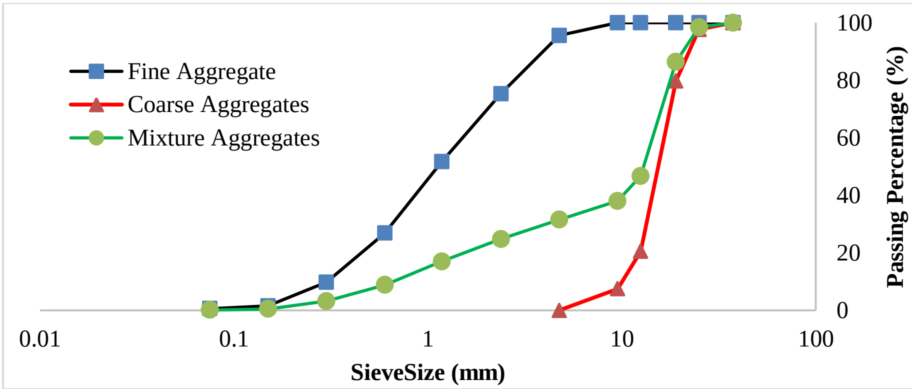

Figure 1: Grading Curve of Aggregates

2.1.3. Water

The clean and pure tap water which is free from organic matters and impurities according to the specification ASTM C 1602/C 1602 M-12 is used in concrete. Water used for mixing of concrete was ordinary tap water.

2.1.4. Admixtures

The admixture used in this study was Chemrite-NN which is a high end product for water content reduction. This admixture is water reducing complying to the ASTM C 494 type C.

2.1.5. Fly Ash

In this research, the type “F” fly ash was used. The F type fly ashes comply with the standards of ASTM C 618.

2.1.6 Polypropylene Fibers

The polypropylene fibers having the lengths of 13 mm, 19 mm and 25 mm were used in concrete. These fibers are according to fiber reinforced concrete specification of standard ASTM C-1116.

2.1.7 Sodium Silicate

In concrete for the activation of fly ash the 4% aqueous sodium silicate (Na_2SiO_3) is used.

2.2 Heating Scheme

To check the temperature effect on the concrete's mechanical properties, the samples were heated in Smith and Foundry shop, Industrial Engineering Department, UET Taxila in the electric furnace at temperature rate of 10 °C/min. The samples were heated at 100, 200, 350, 450 and 650 °C. After heating the samples, when samples attain the specific desired value then samples are kept in furnace to cool down the samples. After cooling to the room temperatures, the samples were then tested to check their mechanical properties.

3. Results and Discussions

3.1. Strength in Compression at Normal Temperature

A total of 6 mix proportions are investigated in this research. All the six mix proportions were tested at normal temperature at 7, 28, 56, 90 and 120 days. The results of compressive strength tests of all these samples at 7, 28, 56, 90 and 120 days are tabulated in table 1. Proper nomenclature has been used in this study: polypropylene fiber reinforced concrete (CPP), inactivated fly ash concrete (CIF), inactivated fly ash polypropylene fiber reinforced concrete (CIFPP), activated fly ash concrete (CAF) and activated fly ash polypropylene fiber reinforced concrete (CAFPP) has been used.

Table 1: Compressive Strength (MPa) at Normal Temperature

Mix Type	Age (Days)				
	7	28	56	90	120
Control	22.85	27.75	29.47	30.12	33.26
CPP	24.1	29.65	35.02	35.73	39.87
CIF	11.37	20.35	27.1	32.97	36.63
CIFPP	13.12	21.04	27.28	33.97	36.68
CAF	23.76	28.98	34.49	37.6	39.48
CAFP	25.85	31.23	39.09	40.39	41.98

The effect of the addition of fiber as compared to without fiber samples demonstrate that the utilization of fibers brought about an enhancement in the compressive strength. This increase in the compressive strength can be justified by the fibers capacity to control the augmentation of cracks, alter the course of cracks, and defer the development of pace of cracks. As shown in the table 1 and in figure 2, the compressive strength of concrete having CAF increases by 52.15 % at 7 days as compared to CIF. The improvement rates were decreased to 29.80 % at the age of 28 days and decreased to 7.22% at 120 days as compared to CIF. Furthermore, at 120 days, improvement in compressive strength was 15.8% higher than the control concrete. The compressive strength improvement can be attributed to aquis Na_2SiO_3 in the solution as the concentration of alkali was maintained by aquis Na_2SiO_3. At early ages the Na_2SiO_3 activator distinctly improved the compressive strength. The effective decimation of the crystalline structure of glossy beads of fly ash and commencement of its initiation results in the compressive strength upgrade at early ages. As illustrated in table 1 that the compressive strength of fibers reinforced concrete was more than the control or inactivated fly ash concrete.

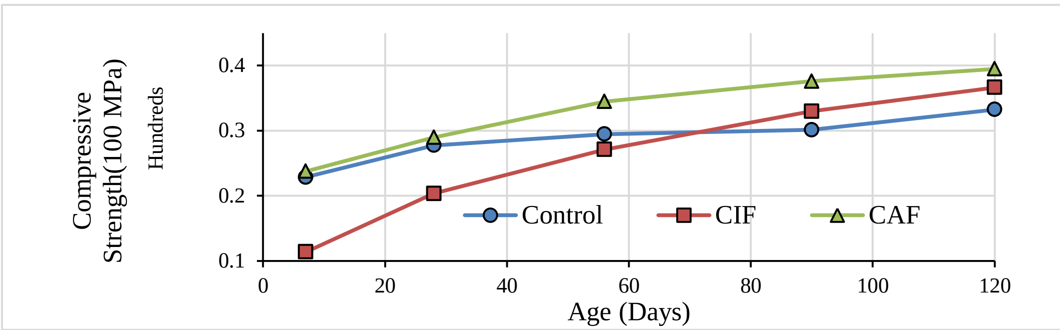

Figure 2: Compressive Strength (MPa) at Normal Temperature for Control Samples

From figure 3, the comparison of inactivated fly ash with polypropylene fiber reinforced concrete to the activated fly ash with polypropylene fiber reinforced concrete is done. It is clear from results that the strength of concrete mixed with polypropylene fiber is improved as compared to control concrete. The results for compressive strength are 49.24 % for 7days, 32.63% for 28 days, 30.21 % for 56 days, 15.90 % for 90 days and 12.62 % for 120 days greater than the polypropylene fiber reinforced concrete with deactivated fly ash. However, the maximum increase in compressive strengths for polypropylene fiber concrete as compared to control concrete were 5.20, 6.41, 15.85, 16.02 and 16.60 % at the age of 7, 28, 56, 90 and 120 days respectively. It can likewise be seen in the table 1 that the impact of polypropylene fibers reinforced concrete was greater as compared to control concrete in the enhancement of the compressive strength. This can be attributed to the higher compressive strength which results in their higher ability in crossing over full scale breaks.

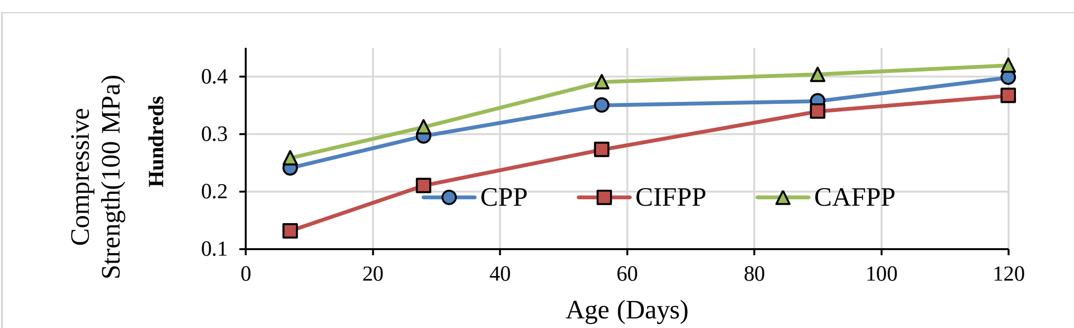

Figure 3: Compressive Strength (MPa) at Normal Temperature for Polypropylene Fiber Reinforced Samples

3.2 Compressive Strength at Elevated Temperature

Compressive strengths of all the mix proportion at elevated temperatures are shown in the table 2.

Table 2: Compressive Strength (MPa) at Elevated Temperature

	Temperature (°C)					
Mix Type	30	100	200	350	450	650
Control	33.26	29.78	22.78	21.37	20.32	17.17
CPP	39.87	34.29	26.31	25.11	23.92	19.94
CIF	36.63	38.06	38.58	36.27	31.55	24.17
CIFPP	36.68	37.48	37.69	35.80	33.30	26.64
CAF	39.48	40.32	40.12	36.80	32.75	26.53
CAFPP	41.98	42.40	39.43	36.67	34.10	30

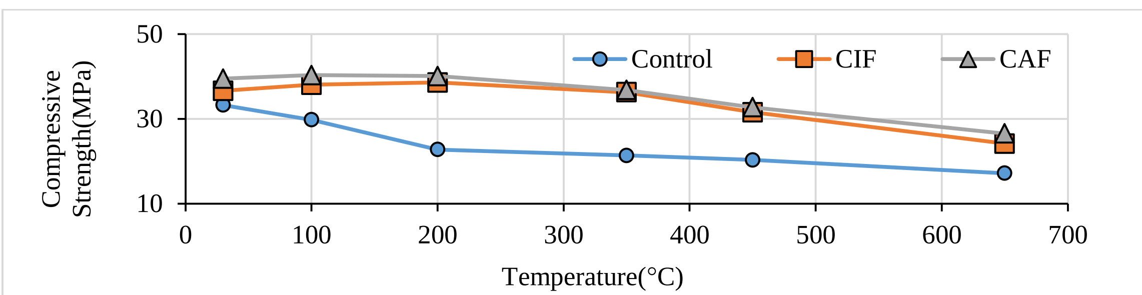

Figure 4: Compressive Strength (MPa) at Elevated Temperature for Control, Inactivated and Activated Fly Ash Fiber Reinforced Concrete Samples

The testing was conducted at the age of 120 days and compressive strength of all three stages of mix proportion at different elevated temperature was noted as shown from figure 4. Generally, exposure to high temperature resulted in loss of compressive strength in case of control and polypropylene fiber reinforced concrete. Within 100 °C, the changes in the strength in compression values were very low when compared to normal temperature strength values.

As shown in the figure 5, at 100 °C (table 2), the decrease in compressive strength for control and CPP were 89.54 and 86.00 % respectively. At 200°C temperature, the remaining compressive strength for Control and CPP were 68.50 and 65.99 % respectively. But at 450 °C temperature, the compressive strength for Control and CPP were reduced by 61.09 and 59.99 % respectively. The compressive strength of control and CPP was nearly following the same pattern as is demonstrated in the graphs of figure 4. It is also of interest to note that when the concrete samples subjected to elevated temperatures i.e. above 400

°C (in the range of 450–650 °C) a rapid loss in compressive strength takes place (figure 4 & 5). It has been noticed that the fly ash concrete compressive strength increases instead of decreasing up to 200 °C. It is a matter of fact that at raised temperature up to 200 °C, the fly ash mixes activated or inactivated, could not attain their maximum strength at age 120 days, because of thermal stimulation and improved strength due to quicker glass breakdown. For example, at 100 °C the residual compressive strength of CIF, CAF, CIFPP and CAFPP were 103.90, 102.13, 102.18 and 101.00 % respectively. After these temperatures the remaining compressive strength decreases because calcium hydroxide ($Ca(OH)_2$) decomposed, especially when temperature ranges between 400–600 °C.

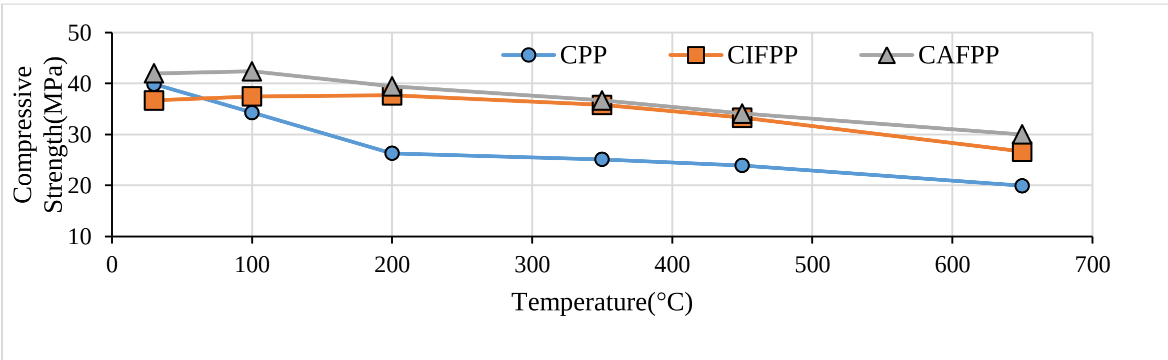

Figure 5: Compressive Strength (MPa) at Elevated Temperature for Polypropylene Fiber Reinforced Concrete Samples

A detailed visual assessment was conducted on the specimens by using PP fiber and control concrete in the presence and absence of activator in fly ash to estimate the observable marks that shows crack and spall after the exposure to elevated temperatures. No significant spalling or visible cracking was observed till the temperature range of 100–650 °C. However, at 650 °C temperature, hairline cracks appeared immensely.

4. Conclusions

- At early ages the strength of concrete reduces when the inactivated fly ash was partially replaced with cement. However, the strength of concrete improved notably at the ages of 56 to 120 days.
- The use of activated fly ash in concrete meaningfully improves the compressive strength at initial days. When the activated fly ash was mixed with concrete, the strength in compression at the age of 07 and 28 days was 52.15 and 29.80 % greater than inactivated fly ash concrete.
- When fly ash is used with or without fibers, it significantly enhances the concrete compressive strength after exposure to temperatures around 100 °C. For example, at 100 °C the strengths of CIF, CAF, CIFPP and CAFPP were 103.90, 102.13, 102.18 and 101.00 % respectively.
- For all elevated temperatures up to 650°C the strength in compression of PP fibre reinforced concrete with activated and inactivated fly ash was greater as compared to control concrete.
- In the whole study a detailed visual assessment was conducted on the specimens by using PP fiber and control concrete using activator and without using activator in fly ash to estimate the observable signs of cracking and spalling after subjected to elevated temperature. There was no significant spalling or visible cracking was observed till the temperature range of 100–650 °C.

5. References

Afroughsabet, Vahid, and Togay Ozbakkaloglu. 2015. "Mechanical and Durability Properties of High-Strength Concrete Containing Steel and Polypropylene Fibers." *Construction and Building Materials* 94: 73–82.

Ali, Faris, Ali Nadjai, Gordon Silcock, and Abid Abu-Tair. 2004. "Outcomes of a Major Research on Fire Resistance of Concrete Columns." *Fire Safety Journal* 39 (6): 433–45.

Alsalman, Ali, Canh N Dang, and W Micah Hale. 2017. "Development of Ultra-High Performance Concrete with Locally Available Materials." *Construction and Building Materials* 133: 135–45.

Biolzi, Luigi, Sara Cattaneo, and Gianpaolo Rosati. 2008. "Evaluating Residual Properties of Thermally Damaged Concrete." *Cement and Concrete Composites* 30 (10): 907–16.

Hefni, Yasmin, Yehia Abd El Zaher, and Mona Abdel Wahab. 2018. "Influence of Activation of Fly Ash on the Mechanical Properties of Concrete." *Construction and Building Materials* 172: 728–34.

Janotka, I, and S C Mojumdar. 2005. "Thermal Analysis at the Evaluation of Concrete Damage by High Temperatures." *Journal of Thermal Analysis and Calorimetry* 81 (1): 197–203.

Kalifa, Pierre, Gregoire Chene, and Christophe Galle. 2001. "High-Temperature Behaviour of HPC with Polypropylene Fibres: From Spalling to Microstructure." *Cement and Concrete Research* 31 (10): 1487–99.

Khaliq, Wasim, and Venkatesh Kodur. 2011. "Thermal and Mechanical Properties of Fiber Reinforced High Performance Self-Consolidating Concrete at Elevated Temperatures." *Cement and Concrete Research* 41 (11): 1112–22.

Uysal, Mucteba. 2012. "Self-Compacting Concrete Incorporating Filler Additives: Performance at High Temperatures." *Construction and Building Materials* 26 (1): 701–6.

Zhang, Peng, and Qing-fu Li. 2013. "Effect of Polypropylene Fiber on Durability of Concrete Composite Containing Fly Ash and Silica Fume." *Composites Part B: Engineering* 45 (1): 1587–94.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March13-14, 2020, Karachi, Pakistan.

Utilization of Natural Fibers in Concrete for Improving Behavior of Non-Structural Elements- A Review

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Abstract

For last few decades, numerous studies have been conducted for using natural fibers in concrete for different non-structural applications. Natural fibers are low cost and can replace synthetic and metallic fibers up to a suitable limit. A lot of research has also been conducted for handling and processing of natural fibers to be used in cement composites. By the addition of natural fibers, it can enhance certain mechanical properties of concrete composites like flexural strength, toughness to fracture, impact resistance. And in some cases, natural fibers have enhanced properties better than glass and carbon fibers. This paper presents a review of recent utilization of natural fibers in concrete for non-structural applications around the globe. First non-structural applications with concrete composites are discussed, and then flaws observed in non-structural concrete members are identified. Aim of this paper is to understand the gap between natural fibers and performance of non-structural elements, for this purpose a comparison of different properties of concrete reinforced with short discrete natural fibers is made along with suggested different non-structural applications by different researchers. Finally, suitable locally available natural fibers to be used in concrete for non-structural applications are recommended.

Keywords

Non-Structural elements, Flexural strength, Natural Fibers.

1. Introduction

Natural fibers are usually used in concrete to enhance the mechanical properties of concrete like, toughness of concrete, flexural strength of concrete etc. Razmi et.al (2017) used jute fiber in concrete to investigate its fracture toughness. For this purpose, he used 0.1, 0.3 and 0.5 percentages by weight and he used 20mm cut length of jute fiber with plain concrete. Specimen shape used for this purpose was semicircular bend. He concluded that by jute fiber enhanced the toughness of concrete. He also found that jute fiber also enhanced other properties like compressive, flexural and tensile strength of concrete.

Many other fibers like coconut, banana sugarcane bagasse etc. have also been studied. Ashes of some natural fibers as fine particles have also been used. Sampath Kumar.et.al. (2016) worked on replacement of cement by using stone dust, rice husk ash and sugarcane bagasse in concrete composites. Specimens were cured for 3, 7 and for 28 days, and performed experiments of tensile and compressive strength. After results he concluded that by addition of rice husk ash, stone dust and sugarcane bagasse increased workability. He also found that by replacement of cement up to 2% can increase strength of composite up to 8%. Ali (2012) reviewed different natural fibers used in concrete as construction material as concrete, cement paste and mortar. He investigated different properties (length, diameter, tensile stress strain and many others) of different fibers like coir, jute, sisal etc. Aziz et.al (1981) presented different researches and development for the utilization of natural fibers in concrete for construction purposes. They described different properties of fibers in concrete and concluded that economical and effective methods for fiber extraction, handling, and conversion into functional materials, concrete mix dispersion, casting, positioning and healing are of great importance. Standard testing protocols for producing quality composites need to be developed and concrete design mix procedures require standardization. Zakaria et.al (2015) investigated properties of concrete composite by reinforcing it with jute yarn, and concluded that by adding 10 and 25mm cut lengths, 0.10 and 0.25% content resulted in maximum tensile, flexural and compressive strength.

In this paper, an effort is being made to understand the gap between material (FRC) properties and performance of non-structural elements. First different non-structural elements will be discussed then flaws identified by different researchers will be observed and by studying different flaws in non-structural elements suitable natural fibers will be recommended.

2. Non-Structural elements

Non-structural elements are those elements which are usually not designed as load bearing structures. They are part of the structure but don't contribute in bearing the load. Non-structural elements in buildings are as partition walls, infill walls, parapet walls etc. As shown in figure 1 non-structural elements made of concrete will be discussed in this study, which includes precast slabs, hollow concrete blocks and concrete roof tiles.

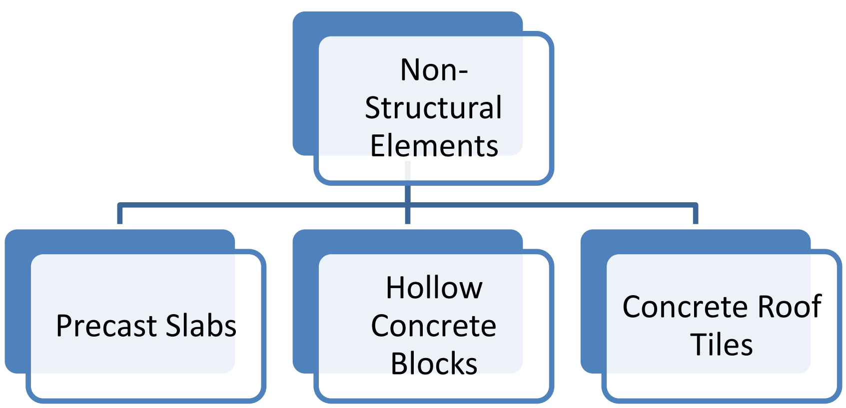

Figure 1: Non-Structural Elements

3. Flaws identified in non-structural elements

After the earthquake disaster in Italy, Braga et.al. (2009) reviewed the performance of non-structural elements. There was mostly damage to non-structural elements (infill and partition walls). Main damages were such as cracks or partition collapse and cracks in infill walls. Socioeconomic casualties were documented as well as human causalities. Hollow brick masonry walls were made as partition walls, which collapsed during the earthquake. The bottom stories were found to be much more affected than above stories.

After the 4 September 2010, Darfield (Canterbury) earthquake and subsequent aftershocks, Rajes & Dhakal (2010) described damage to non-structural components in buildings. Damage to the structural structure was also observed, but there was major loss of non-structural elements. With subsequent aftershocks, the duration and intensity of the earthquake, the damages to non-structural elements were reported to have increased. Brick chimneys, parapets, ceilings, facades, infill walls and windows were the components that were observed to have been damaged. In addition, it was also found that commercial buildings, residential buildings and offices have suffered poor damage.

Shaikh, & Feile (2004), used a double tee flange connector to perform pre-cast slab experiment. Slab and tee flange behavior for various aspects such as monotonic connections for horizontal shear without tension, tension-free cyclic horizontal shear, and two more. He found in his study that spalling occurred under the concrete slab due to shear. At the bottom of the slab a crack developed on loading, then corner chip off occurred, due to spalling. He was unable to extract two of his experimental values due to this cracking. He recommended using concrete fiber to increase strength.

Su, R. K. L et.al. (2005) carried out a case study on the lateral stiffness of the building caused by the building's non-structural members. Three high rise buildings (SB), TT Tsui Building (TTT) and Typical Harmony Blocks (THB) were observed in this study. Mostly, he observed that the building's stiffness was enhanced by the installation of infill walls (usually made of concrete blocks and tiles) in molding. When these blocks became in contact with seismic beam it increased the building's stiffness.

Rajeev, P. et.al (2016) studied behavior of thermal loading on concrete roof tiles, a 3D model of finite element for typical roof tiles in Australia was created to investigate the thermal cracking in them and width of the cracking. Temperature varied on both top and bottom side, while performing experiment they found that tensile stresses in x and y both directions increased with the increase in difference between top and bottom side of tiles, and it seemed to rise and was finally leading to the risk of cracking.

Sarhat et.al. (2014) performed experiments on ungrouted hollow concrete blocks to develop a formula to predict their compressive strengths. The data obtained from tests were also used to develop a model and this model was also compared with codes. Factors that affect the ungrouted masonry’s compressive strength were also discussed. In these factors, it was said that tensile cracking in prism was also associated with compression failure.

So from all these literature, it is found that main flaw in pre-cast slab is spalling, in hollow blocks and roof tiles have deficiency in tensile strength.

4. Preventive measures take to improve behavior of non-structural elements

Studies around the world have been conducted to find the possible and effective approach to improve non-structural elements in existing buildings.

Palerm et.al. (2010) summarized the facade technology available in New Zealand and around the world to propose a framework for the classification of facade systems for the emerging seismic engineering concept of performance-based seismic design will be introduced in relation to the performance of the façade using this classification framework.

They said that façade system can be classified into three types: infill, cladding and last is their combination. Problems in façade system and their solutions are discussed in this paper.

Almusallam et.al (2007) said that fiber-reinforced polymers (FRP) are considered to be one of the most appropriate retrofit solutions for reinforcing masonry infill panels in RC frames. Test results show that the use of glass FRP sheets as reinforcing materials enhances the infill panels, improves their strength and ductility and converts wall into single unit.

5. Use of natural fibers in concrete for improving non-structural elements behavior

Natural fibers are being used to enhance the properties of concrete composites. Numerous studies have been conducted and different types of natural fibers have been used in the enhancement of concrete composite.

5.1. Use of natural fibers in concrete composites for enhancing properties of concrete

Mohaiminul Haque et.al (2019) performed a study in which, used coconut fibers as a concrete preparation reinforcement component. The composite contains natural aggregates of 70 % and concrete aggregates of 30% recycled. Based on the experimental work, he concluded that adding a small amount of coconut fiber (2%) would increase the compressive strength of recycled concrete by about 79%.It was found that the same fiber percentage (2%) increased the tensile strength of concrete by approximately 73.16%. It was also noticed that this method was a convenient way to use the recycled concrete aggregate with a goal of achieving better sustainability. Zakaria et.al (2015) used jute yarn to investigate the mechanical properties of concrete. He observed the compressive, flexural and tensile strength of concrete composite by different parameters such as shape (he used prism, cube and cylinder), cut lengths (10, 15, 20 and 25 mm) and volume content of jute yarn (0.1%, 0.25%, 0.5% and 0.75%). After experimentation, he concluded that for cut lengths of 15 mm jute yarn and 0.10 percent of jute yarn content and for concrete mix of 1:1.5:3 or 1:2:4 respectively gave the maximum value for compression which was 33% and 12% improvement in strength. For flexural 23% increase in strength was obtained for a ratio of 1:1.5:3 of 15 mm length of 0.10% content of jute yarn and 16%increase was for ratio of 1:2:4 of 10 mm yarn length and 0.25% content. As for tensile, the yarn content was 0.10 and 0.25% for 10 and 15 mm.

Properties of four different types of coir, sisal, jute and Hibiscus cannebinus with four different fiber contents such as 0·5%, 1·0%, 1·5% and 2·0% by weight of cement were studied by Sivaraja et.al (2010). The experiments were carried out using repeated projectile, and the strength of the material was determined based on the parameters that were impact resistance, residual effect ratio, crack resistance ratio, and ultimate fiber quality. The test results indicated that coir fibers absorbed more energy, i.e. 253·5 J at a fiber content of 2 percent at a length of 40 mm. Sugarcane bagasse ash, rice husk ash and stone dust were used in concrete mix by K.S. Kumar et al. (2016) with partial cement replacement. He studied the properties of concrete composites after curing of 3, 7 and 28 days. He performed two types of tests on concrete models, one of which was split-tensile strength and the other was compressive strength test. He concluded that by replacing 6% cement and adding 2% of each material, the compressive strength of the specimens could be increased, and by replacing 2% cement with composite materials, the strength could be increased by up to 8%.

6. Conclusion

Review of different studies was done to find the flaws present in non-structural elements. The main flaws found in non-structural members (made of concrete) were: spalling in pre-cast slabs, compressive failures due to tensile cracking in hollow concrete blocks and thermal cracking due to increase in tensile stresses in x and y direction. Then different natural fibers are reviewed to see their potential to be used in concrete for enhancement of flaws in non-structural members. It was found that jute fiber and coconut fiber had potential to complete the deficiencies present in non-structural members.

7. Acknowledgment

The authors would like to acknowledge the efforts and help of Hamza Ali Shah, Waqar Ahmad and anonymous reviewer.

8. References

Ali, M. (2009). Natural fibres as construction materials. Materials and Technologies (NOCMAT 2009), 6, 9.

Almusallam, T. H., & Al-Salloum, Y. A. (2007). Behavior of FRP strengthened infill walls under in-plane seismic loading. *Journal of Composites for Construction*, *11*(3), 308-318.

Aziz, M. A., Paramasivam, P., & Lee, S. L. (1981). Prospects for natural fibre reinforced concretes in construction. International Journal of Cement Composites and Lightweight Concrete, 3(2), 123-132.

Haque, M., Ray, S., Rahman, M. M., &Washif, m. properties of recycled aggregate concrete with coconut fibers.

Kumar, K. S., Praveen, U. M., Prathyusha, A., Akhila, V., &Sasidhar, P. (2016). A Comprehensive Study On Partial Replacement of Cement with Sugarcane Bagasse Ash, Rice Husk Ash & Stone Dust. *International Journal of Civil Engineering and Technology*, *7*(3), 2016.

Palermo, A., Pampanin, S., Baird, A., &Riccio, P. (2010). Focusing on reducing the earthquake damage to non-structural components in buildings. In *NZSEE Conference*.

Rajeev, P., Sanjayan, J. G., &Seenuth, S. S. (2016). Assessment of thermal cracking in concrete roof tiles. *Materials & Design*, *107*, 470-477.

Sarhat, S. R., & Sherwood, E. G. (2014). The prediction of compressive strength of ungrouted hollow concrete block masonry. *Construction and Building Materials*, *58*, 111-121.

Shaikh, A. F., &Feile, E. P. (2004). Load testing of a precast concrete double-tee flange connector. *PCI journal*, *49*, 84-95.

Sivaraja, M., Velmani, N., & Pillai, M. S. (2010). Study on durability of natural fibre concrete composites using mechanical strength and microstructural properties. *Bulletin of Materials Science*, *33*(6), 719-729.

Su, R. K. L., Chandler, A. M., Sheikh, M. N., & Lam, N. T. K. (2005). Influence of non-structural components on lateral stiffness of tall buildings. *The Structural Design of Tall and Special Buildings*, *14*(2), 143-164.

Zakaria, M., Ahmed, M., Hoque, M. M., &Hannan, A. (2015). Effect of jute yarn on the mechanical behavior of concrete composites. *SpringerPlus*, *4*(1), 731.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Adoption of Building Information Modeling (BIM) in Construction Industry of Pakistan: Benefits and Barriers

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Abstract

The increasing complexity in nature of construction projects calls for efficient management in order to avoid losses, delays and other unwanted scenarios. Therefore, a lot of techniques are being developed and employed to obtain high quality of project within limited resources. Building Information Modeling (BIM) is one of such techniques that are increasingly employed in global construction industry because of the benefits it offers suc as: cost, material and time estimates, virtual representation of the project, improved collaboration and helps efficient decision making. BIM integrates various disciplines such as Mechanical, Electrical, Plumbing, Construction etc. by effective communication and helps in analyzing the project systems for constructability. BIM is helpful in proper visualization of project even before its commencement; hence it eliminates the construction clashes which ultimately help in avoiding waste generation, cost over runs and time over runs. Despite of such key benefits offered by BIM, the adoption of BIM in construction industry of Pakistan is very low and there are various barriers which contribute to its slow implementation. This research is therefore aimed to bring out the benefits and barriers of BIM so that the potential of BIM can be explored and utilized and in addition to that; the causative factors for its slow implementation may be identified and eradicated. The ranking of identified benefits and barriers of BIM adoption was done through a questionnaire survey conducted with professional respondents of construction industry. The results revealed that as BIM is a new talk of the town, there is a lack of skilled professionals, lack of government interest, lack of client demand etc. Because of unavailability of skilled BIM professionals, high cost of human resource is also a factor that hinders the adoption of BIM. Individuals in the said industry are very much reluctant in changing the organizational culture and conventional methods being used. Moreover, the initial setup of BIM is difficult, and costs associated with its implementation are very much high. These are few of the barricades in BIM implementation identified in this study.

Keywords
Construction Projects, Building Information Modeling, Cost Overruns, Time Overruns, Benefits, Barriers.

1. Introduction

Since past few decades, the construction industry of Pakistan has been less productive due to the casual management of projects with conventional methods being employed. This improper management of projects in construction sector has consequently led to cost and time over runs of projects. Building Information Modeling (BIM) is one out of the most efficient approaches developed to counter all such factors leading to lesser productivity in construction sector. BIM helps in reducing the cost of project by eliminating waste generation, identifies clashes in design phase rather than execution phase, creates efficient schedules and increase collaboration between stakeholders of the project **(Siddiqui, F.H. et. al, 2019)**. BIM is an approach gaining popularity due to the benefits it offers such as visualization of project even before its construction, calculation of quantities of materials, increasing the quality of project etc. **(Rokooei, S. 2015)**. It reduces waste generation and reworking that leads the project to be completed within the resources such as time, cost and available material. BIM also helps in implementing and enhancing safety protocols in construction **(Akram R. et. al, 2019)**.

The level of implementation of BIM is increasing day by day in well-developed countries while there are numerous barriers which hinder its use in developing countries like Pakistan. In a research study conducted in USA, it was concluded that 8 out of every 10 projects employ BIM using Autodesk Revit tool. The use of BIM has also increased in construction sector of UK and Australia, **(Becerik-Gerber, B. et. al, 2010)**. In India, 8 out of 25 respondents were using BIM in their projects which make only 32%. The Autodesk Revit and Google-SketchUp were the most used tool for BIM in India **(Luthra, A., 2010)**. For countries like Finland, Denmark and Hong Kong, the Architecture, Engineering and Construction (AEC) Industry is shifting from 2D Computer-Aided Design to 3D Modeling of Structures. In Hong Kong, only one Architectural firm had around 70% of its current projects using BIM **(Chan, C. T, 2014)**. It was concluded from the study conducted in Denmark that around 50% of the architects, 29% of clients and 40% of engineers were somehow using BIM in their projects in year 2006. The use of BIM in Finland was promising as concluded by a survey conducted in year 2007. As per results, 93% of Architectural Firms and 60% Engineering Firms were using BIM in their projects **(Wong, A. K. D., et. al, 2009)**. With every passing year, the extent of implementation of BIM is expanding worldwide. According to a research carried out by Autodesk Revit, it has been observed that 18% of owners used BIM in 2009 which increased to 30% in 2011, and up to 44% in 2014 **(Autodesk, 2016)**. This study, therefore, aims to identify the potential offered by BIM and the barricades in its implementation in construction sector of Pakistan.

2. Aim and Objectives

The aim of this research is to reveal the importance of adoption of Building Information Modeling (BIM) in Construction Industry of Pakistan. Following specific objectives were set to achieve the aim of study:

1. To identify the benefits of BIM in AEC Industry of Pakistan.
2. To identify the barriers in implementation of BIM in AEC Industry of Pakistan.

3. Literature Review

The available literature regarding BIM carries numerous benefits and barriers of BIM implementation in AEC Industry such as BIM helps in reducing costs and duration of project. Moreover, it helps in creating effective collaboration between stakeholders hence increases quality and performance of project. It also gives accurate estimates of quantities which helps in reduction of waste generation in construction **(Rokooei, S., 2015)**. Authors in a study carried out to reveal benefits of BIM concluded that BIM is helpful in avoiding disputes in construction and making efficient decisions. It also helps in reducing reworking and making the construction and scheduling efficient. BIM also helps in prefabrication of building elements **(Liu, S. et. al., 2015)**. BIM provides enhanced customer services through visualization of project even before its execution. It also reduces human resource and waste generation occurring from clashes identified during construction as it identifies the upcoming clashes between structural elements even before the construction of a project **(Yan, H., et. al., 2008)**. Moreover, the BIM also reduces the cost and time of the project which is quantified in **(K. Barlish, et. al ,2012)**. The BIM improves the project designs by removal of clashes between different disciplines **(Krystallis, Ilias et. al, 2019)**.

Numerous other advantages of BIM have been identified by several authors such as; BIM reduces change orders in construction projects, allows simultaneous work by multiple disciplines, provides new revenue and business opportunities, provides accurate and consistent drawing sets and helps facility manager by providing facility data that can be used for safe, healthy and effective work environment **(Becerik-Gerber et. al., 2010)**. Other benefits of BIM include, improved construction safety, indication of building failures, Light, Mechanical and Acoustics analysis can be performed using BIM. Moreover, energy consumption of project can be analyzed etc. **(Rokooei, S., 2015; Becerik-Gerber, et. al., 2010)**. BIM also reduces construction claims, makes smarter and faster changes and helps the architects to analyze the location of building **(Lindblad, H., 2013)**.

Despite of these benefits offered by BIM, its adoption is very slow and there are numerous barriers that hinder its implementation in construction sector. Few of the top barriers are: Lack of skilled personnel, high cost of implementation, legal and security issues, lack of national standard etc. **(Rokooei, S., 2015; Liu, S. et. al., 2015)**. Organizational behavior such as organizations are reluctant in implementing new technologies, lack of interest of government and increase in the cost of human resource due to lack of professionals also cause hindrance in the adoption of BIM **(Liu, S. et. al., 2015)**.

There is a misconception that BIM wastes time and cost which hinders the adoption of BIM hence there is lack of a case study as an evidence of financial benefits offered by BIM. Another barrier in adoption of BIM is that people refuse to learn and adopt it **(Yan, H. et. al, 2008)**. Other barriers include lack of client demand, lack of IT infrastructures, difficult and expensive initial setup of BIM **(Chan, C. T., 2014)**. The need of complex software tools is also hindering the successful implementation of BIM in construction industry **(Lindblad, H., 2013)**.

4. Research Methodology

A multi-step research methodology was adopted in this study. Initially, literature review was conducted to identify the potential benefits and barriers associated with adoption of BIM in construction sector. Unstructured interviews with several relevant professionals and stakeholders were conducted for inclusion of local factors in questionnaire. The questionnaire survey was then conducted through Emails, Google Forms and Hard Copies of questionnaire propagated among construction professionals identified from database of Pakistan Engineering Council (PEC). The data collected was then analyzed through software SPSS version 24.0. A five-point likert scale was used to assess the significance level of each of the factors, in which 5 = Very Significant and 1 = Not Significant. Consequently, after detailed analysis of obtained data, necessary conclusions and future recommendations were made. The average indexes were calculated through SPSS by input of all the data gathered through questionnaire based on Likert scale. The average index method was chosen so that the impact of each factor can be seen on the scale of 5 gathered from all the results of questionnaire. The formula used for calculating the values of Average Indexes is adopted from **(Akhund, M.A. et. al, 2015)** as it is simple to understand and use.

$$\mathbf{AI} = \frac{\sum(\mathbf{5X1 + 4X2 + 3X3 + 2X4 + 1X5})}{\sum(\mathbf{X1 + X2 + X3 + X4 + X5})}$$

where;

X1 = Number of respondents for scale 1
X2 = Number of respondents for scale 2
X3 = Number of respondents for scale 3
X4 = Number of respondents for scale 4
X5 = Number of respondents for scale 5

The flow chart of research methodology is as under in Figure 1.

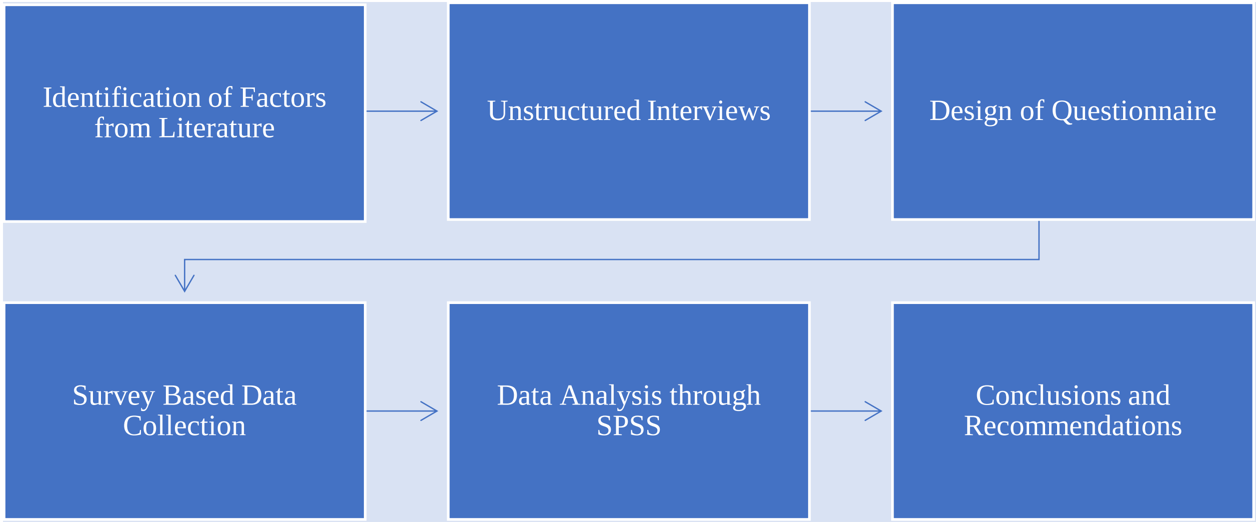

Figure 1: Research Methodology Flowchart

5. Data Collection

In total, 300 questionnaires were distributed and 110 were received out of which 90 were valid and remaining 20 were null and void as the response was incomplete. The demographic factors of respondents participating in this study are given in figures below. The figure 2 shows the profession of the respondents which depicts that it filled by different types of professionals in relation to the benefits of BIM.

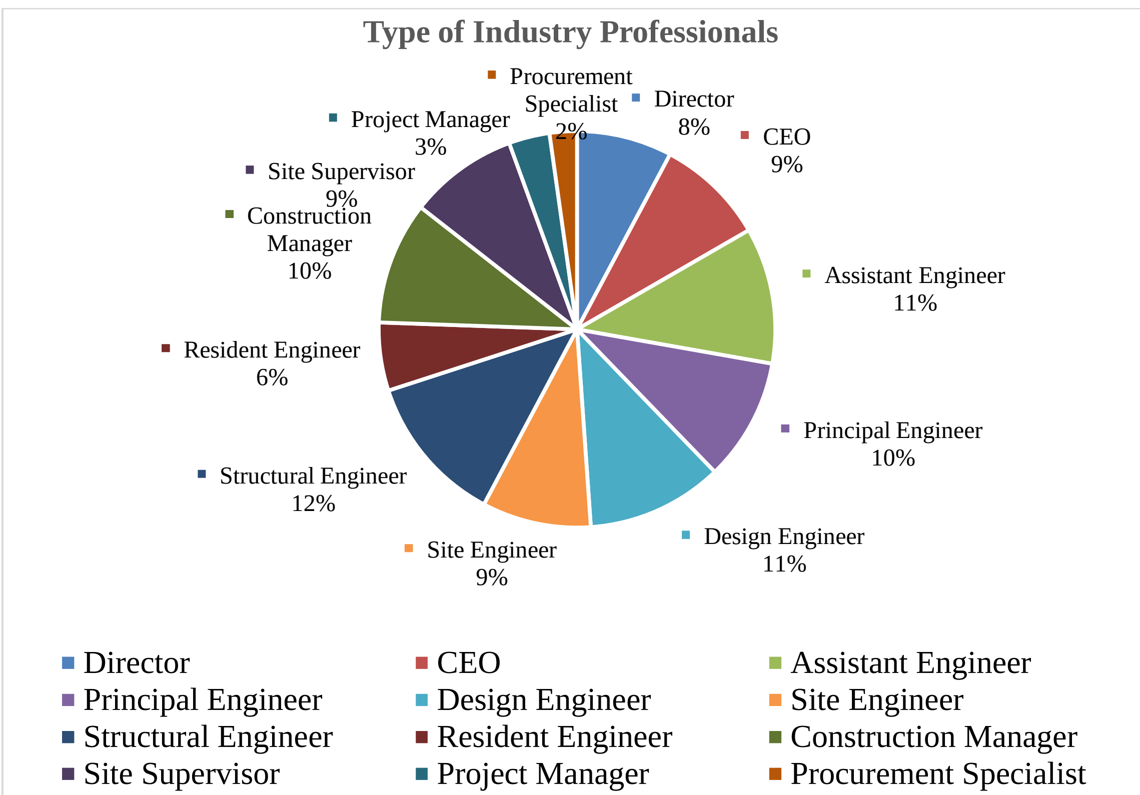

Figure 2: Demographical Factors of Respondents Profession

The figure 3 states the experience of professionals had which was helpful in identifying the barriers of BIM in Pakistan.

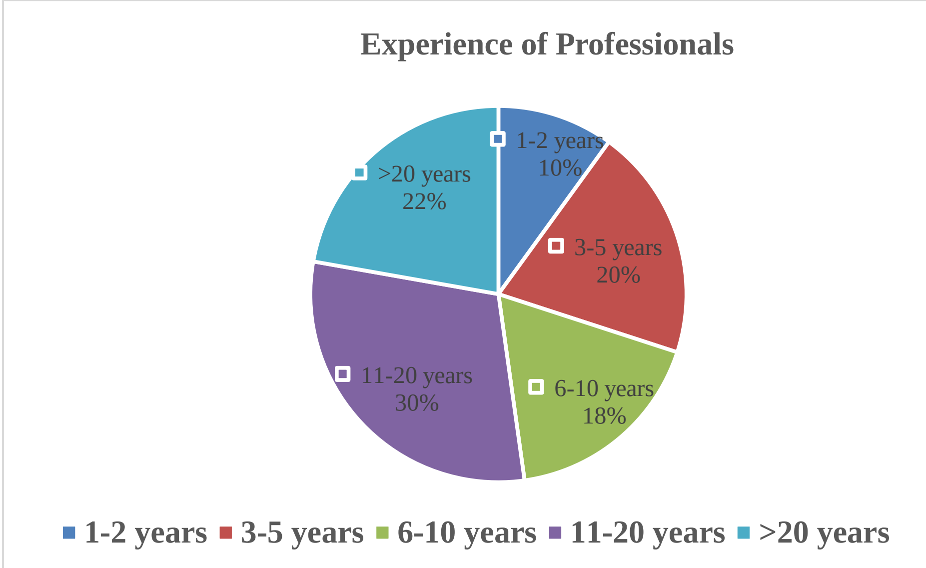

Figure 3: Demographical Factors of Respondents Experience

The figure 4 quotes the percentage of respondent's education which are working in different sectors as shown.

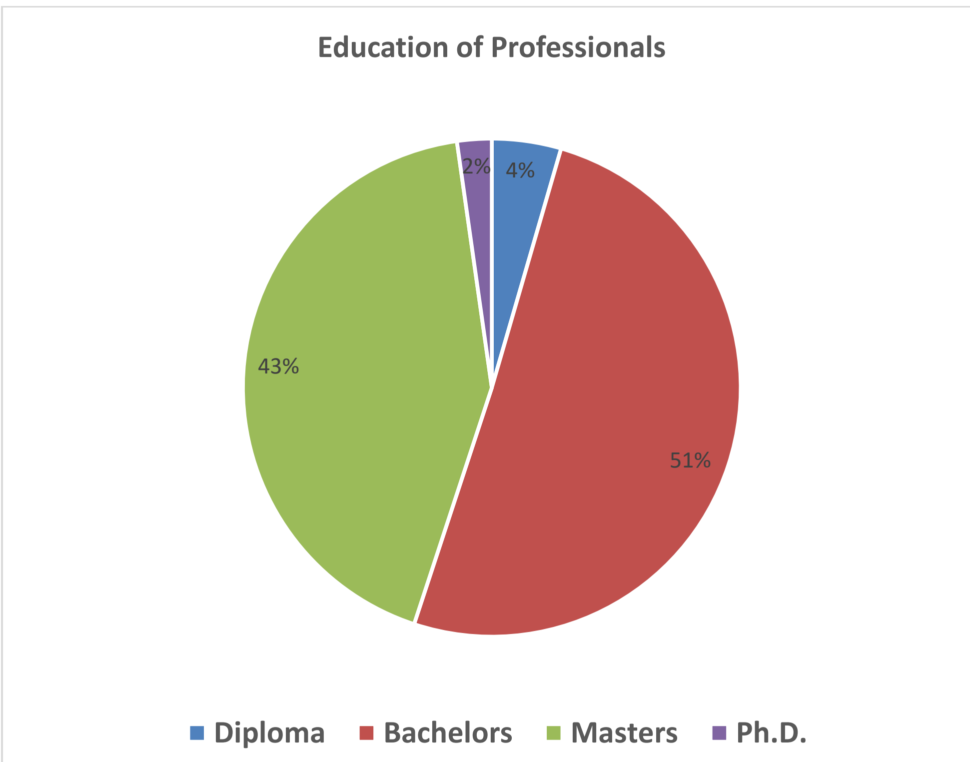

Figure 4: Demographical Factors of Respondents Education

6. Results and Discussions

After the detailed analysis of data obtained from skilled professionals, the benefits of BIM are listed below in figure 5:

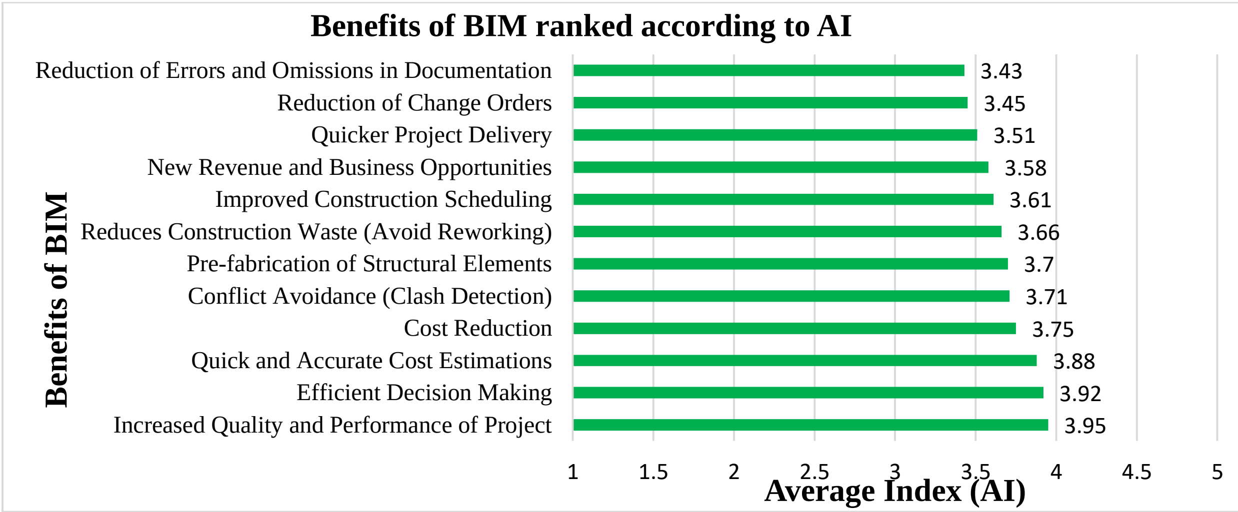

Figure 5: Benefits of BIM ranked with respect to Average Index

The results validated the numerous benefits of BIM such as increase in quality and performance of project, quick and accurate cost estimations, reduction in cost, increase in business opportunities, avoidance of clashes and conflicts, reduction of construction waste and reduction of change orders, errors etc. In continuation to this, the research study and questionnaire survey also identified some of the key barriers hindering the adoption of BIM in construction sector of Pakistan. The barriers are presented as graph in figure 5 which shows results according to the survey results.

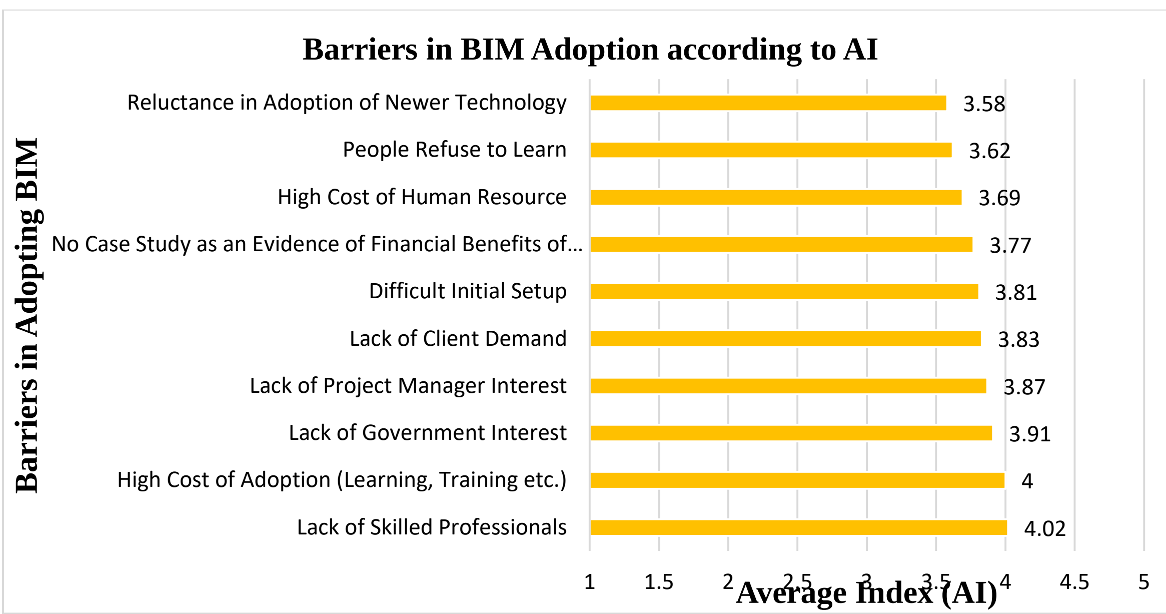

Figure 6: Barriers in BIM Adoption

The results indicate that as BIM is a modern technique, there is a lack of skilled professionals, lack of government interest, lack of client demand etc. Because of unavailability of skilled BIM professionals, high cost of human resource is also a factor that hinders the adoption of BIM. Individuals in the said industry are very much reluctant in changing the organizational culture and conventional methods being used. Moreover, the initial setup of BIM is difficult, and costs associated with its implementation are very much high. These are few of the barricades hindering the adoption of BIM in construction sector.

7. Conclusions:

1. From the results, it is observed that BIM is beneficial in increasing the quality and performance of the project and with the latest tools BIM provides us accurate schedule of materials and their cost estimates.

2. The results can also help us understand that BIM could be a process in reducing cost of the project by reducing clashes with better integration of different disciplines.

3. BIM maximizes the project efficiency with better project scheduling and detection of clashes which reduces the waste and time for completion of project.

4. The biggest challenges right now in adoption of BIM is lack of skilled professionals for which recommendations are given in recommendation section of this paper.

5. BIM is often times associated in with higher cost for setup and human resource which can be reduced by introducing a policy at government level which is further explained in explanation section of the paper.

8. Future Recommendations

Based on the findings the study suggests that:

1. The academia should include courses related to Building Information Modelling in both graduate and Undergraduate programs so that graduates from both programs can be made familiar with both programs and can work more on that in their professional field.

2. In Pakistan, the professionals of Architecture Engineering and Construction (AEC) are still unaware about the processes of Building Information Modelling, so there should be awareness sessions conducted by Pakistan Engineering Council (PEC) and Engineering Universities for professionals and students. The awareness can be given through seminars, webinars and workshops, and as a benefits CPD points should be awarded by PEC so that Engineers can join in good numbers.

3. Keeping the benefits of BIM in consideration, a framework should be implemented at departmental level of different government institutes through which BIM can be made part of contract and design process which can result in increased productivity of construction industry of Pakistan.

4. In this research a questionnaire was used to get information from the BIM users about the factors identified from the literature review, but case studies of BIM based buildings in Pakistan are not included in this study. Hence, in future, it is recommended that various case studies should be reviewed for further validation of the claims of BIM increasing productivity of Construction Industry.

5. The research done by academia should be given practical approach by the industry which would increase efficiency of the projects, for this purpose better industry academia linkages are recommended to see the benefits of BIM on live projects.

References

Akram R. et. al, (2019), "Exploring the role of building information modeling in construction safety through science mapping", *Safety Science* 120 (2019), 456-470.

Akhund M.A., et al., (2015), "Time Overrun in Public Sector Construction Projects of Developing Countries: Case Study of Pakistan"., *2nd International Conference on Emerging Issues in Management and Economics. 09-10 October 2015.*

Autodesk (2016), "The Business Benefits of BIM for Owners".

Becerik-Gerber, et. al, (2010), "The perceived value of Building Information Modeling in the US building industry", *Journal of Information Technology in Construction (ITcon)*, 15 (15), 185-201.72.

Chan, C. T. (2014), "Barriers of Implementing BIM in Construction Industry from the Designer's Perspective: A Hong Kong Experience", *Journal of System and Management Sciences*, 4(2), 24-40.

K. Barlish, et. al, (2012), "How to measure the benefits of BIM - A case study approach" *Automation in construction", 2012, vol. 24., pp. 149-159*

Krystallis, Ilias et. al, (2019), "Towards a methodology for quantifying the benefits of BIM." *In Proceedings of 2019 European Conference on Computing in Construction (EC3). European Council on Computing in Construction (EC3), 2019.*

Lindblad, H. (2013), "Study of the implementation process of BIM in construction projects", *Thesis of Master of Science, Department of Real Estate and Construction Management, Architecture and Build Environment, 2013.*

Liu, S. et. al, (2015), "Critical Barriers to BIM Implementation in the AEC Industry", *International Journal of Marketing Studies, 7(6), 162.*

Luthra, A. (2010), "Implementation of Building Information Modeling in architectural firms in India", *Thesis submitted to Purdue University for the award of degree of Master of Science in Technology*.

Rokooei, S. (2015), "Building Information Modeling in Project Management: Necessities, Challenges and Outcomes", *Procedia-Social and Behavioral Sciences*, 210, 87-95.

Siddiqui, F.H. et. al (2019), "Barriers in Adoption of Building Information Modeling (BIM) in Pakistan's Construction Industry", *Indian Journal of Science and Technology*, Vol 12(25), DOI: 10.17485/ijst/2019/v12i25/142325, July 2019.

Wong, A. K. D., et. al (2009), "Comparative Roles of Major Stakeholders for the Implementation of BIM in Various Countries", *Proceedings of the International Conference on Changing Roles: New Roles, New Challenges, Noordwijk Aan Zee, The Netherlands pp. 5-9.*

Yan, H., et. al (2008), "Benefits and Barriers of Building Information Modelling". *Proceedings of the 12th International Conference on Computing in Civil and Building Engineering (ICCCBE-XII) and International Conference on Information Technology in Construction, 2008. pp. 1−6*.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

SCHEDULE PERFORMANCE IN DAM CONSTRUCTION PROJECTS OF SINDH

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Abstract
Water is essential source for economic activitites. According to 2017 census 'Sindh is most urbanized province of Pakistan'. It is the main reason which increse the battling for water in region and also it can not easily acces for productive and consumptive use.To counter it dam construction is a significant plan of action. However, literature in this area indicates that there exists schedule performance related issues in dam construction projects of Sindh. Therefore, this research in mainly carried out to identify the main causes affecting schedule performance in dam construction projects of Sindh. Eighty causes were identified from an extensive literature review. Furthermore, interviews were conducted from industry experts involved in dam construction projects. The outcome of study reveals that 'Financial problems faced by client', 'Delay in government authorities' permits related to project', 'Delay in decision making by client/consultant', 'Delay in payment by client', 'Suspension of work', 'Contractor's financial difficulties in case of cash flow', 'Improper planning and scheduling', 'Delays in payment to subcontractor/ suppliers' are some of the important causes affecting schedule performance in dam construction projects of Sindh. Based on finding of the research, this study also suggests some measures for improving schedule performance of dam projects.

Keywords
Schedule Performance, Dam Construction, Sindh.

1. Introduction

In economic stabilization of any country, construction industry plays crucial role. A huge section of socio-economic growth depends on inclination or declination of construction industry. According to economic survey of 2018-2019, construction activity has declined by 7.57% as compared to last year due to favoring free enterprise construction associated with other expenditures in different economic activities [1]. Hence, there is immense need that the projects should be successful. A project is successful if its completion is under a limit of budgeted cost, scheduled time and within required quality. Cost, Time and Quality are the principle apex points of Iron triangle of project management and obviously they are directly dependent on each other. Moreover, Cost and Time are the two important aspects of project life cycle that depict the full scenario of project and also the lucrativeness of any construction of project [2]. Unfortunately, construction projects are facing poor cost and time performance. Moreover, the Poor cost performance is known as estimated budget at which project objectives still not achieved Cost performance is an important aspect in success of project which should be strictly followed [3]. Poor time performance is expansion in duration of project beyond scheduled dates of delivery of project [4]. Because of the several factors these face the high level of variation as some research shows that at global point of view 90% of projects undergo 28% of cost increment with respect to estimated cost. In USA only 16% of projects were completed within estimated cost, scheduled time and required quality [2].

According to the public accounts committee (PAC), Islamabad report (2017) mentioned that "1000 federally funded schemes worth Rs7.9 trillion were delayed and cost performance in some cases swelled more than the original cost of projects" [5]. Among provinces, Sindh is the 2nd largest province of Pakistan and after Punjab, Sindh is the most populated province of Pakistan according to census 2017 [6]. Sindh contributes about 30% to 32.7% in GDP of Pakistan, and a large portion of population of Sindh belong to agriculture and it use 69% of water to fulfill other requirements of present as well as future it is mandatory to store water and decrease the waste of water [7]. Therefore, since year of 2000 Sindh Government involved in construction of considerable numbers (large and small) dams. Some of dams completed and some are still under construction, according to province's Annual Development Plan (ADP) "50 dams will be constructed out of which 28 dams are completed and 22 dams are under construction". Among these dams of Sindh, Nai-Gaj dam project is a known for most affected project due to poor time and cost performance [8]. According to a DAWN report, Nai-Gaj dam project is long-delayed and badly affected by cost overrun. The dam was scheduled to be completed in April-May 2015 [9]. In context of the above, it is imperative to address the issue of poor schedule performance of dam construction projects in Sindh.

2. Literature review

A survey through questionnaire on time performance of construction projects of Sindh, depicted that design changes, need of extra level quality, hindrance due to government, relation between client and contractor, late delivery of materials and many other causes need more attention to avoid the delay in construction projects [10]. Many articles and studies conducted on schedule performance of construction projects, both internationally and locally, have been reviewed. As a preliminary study was performed in Malaysia which focused on life cycle of project and evaluating the risk of numerous causes in time and cost performances in construction ventures. The study highlights project life cycle related risk factors [11]. Through an excessive literature review a theoretical framework was established by a Malaysian researcher in which 48 causes of delay and 38 causes of cost increment were identified to help construction practitioners to avoid the involvement of these causes [12]. Further, in Egypt another research identified the flaws in time and cost performance in construction projects. The outcome of research depicted the most five causes that affect the time and cost performance which are lack of financial resources for contractors, substandard feasible planning, unsatisfactory duration and cost control, low efficiency of labors, ineffective communication across parties, extra load from client and rise of materials price [13]. A research was done in Australia on water infrastructures to find out delay causations. Most dominant factors identified in this study were: availability of considerable quantum, quality of lessons from similar projects, logistic issues and project complexity and expanse, credibility of planning techniques, uncertainties around implementation, environmental constraints and limited interoperability between disciplines [14]. A study was conducted on time performance in building projects of Pakistan and identified that financial problems faced by contractors, contractor`s inexperience, weather impacts, late delivery of material, mistake in design, lack of skillful labor, incompetent subcontractor and mistake in estimation were main causes of poor time performance in projects [15]. A research carried out on highways projects in Sindh mainly focused on cost performance. High causative attributes identified were holding payment from client side, client unnecessary interruption, low management in contract, variation in scope of objectives, monetary problems of client, lag in decision taking [16]. Based on extensive literature review, the mapping of the causes influencing schedule performance in different construction projects is given in table 2.1.

Table 2.1: Mapping of Causes Influencing Schedule Performance

			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
S.No:		**CAUSES INFLUENCING SCHEDULE PERFORMANCE OF DAM CONSTRUCTION PROJECTS**	**Soomro, F.A. -Jan 19**	**Bansal, V. May-19**	**JohnsoR.M. Nov-18**	**Rahman, I. A. -18**	**Karami, H. Jan-18**	**Sohu, S. Feb-17**	**Ullah, K. Nov-17**	**Akhund, M.A. -17**	**Akal, A.Y. Nov-17**	**Amoudi, O. may -17**	**Memon, N.A. April -16**	**Senouci, A. Dec- 16**	**Aziz, R.F. Mar-16**	**Shibnai, A. Sept-15**	**Vaardini, U.S. Sept-15**	**Ali, T.H., June -15**	**Marzouk, M.M.-14**	**Ismail, I. Dec-13**	**Shibnai, A Oct- 13**
1	**Contractor Related**	**Poor site management and Supervision**	□	□	□	□		□	□	□		□	□	□	□	□	□	□	□	□	□
2		**Improper planning and scheduling**	□	□	□	□	□	□	□	□	□	□	□	□	□		□	□	□	□	□
3		**Contractor insolvency or financial difficulties cash flow**	□	□	□	□		□	□	□	□	□			□		□	□	□	□	
4		**Lack of experience of contractor**	□	□				□	□			□	□		□	□		□	□		□
5		**Reconstruction of completed work \Defective work**	□	□	□	□		□	□	□		□	□	□		□	□	□	□	□	□
6		**Delays in payment to subcontractor/ Suppliers**							□						□					□	
7		**Incompetent Subcontractor**	□		□	□		□	□				□	□			□			□	
8		**Contractors delay in execution of work**		□		□							□				□			□	
9		**Conflicts in work schedules with contractors and sub-contractors**		□		□				□		□						□			
13		**Poor performance of contractors**								□							□				
14		**Inaccurate cost estimation and Quantity estimation**		□	□	□						□	□					□		□	□
15		**Dispute between contractor and consultant**		□								□									
16		**Delay in site mobilization**					□	□		□						□		□	□		
17		**Shortage of skill laborers**	□	□	□	□		□	□	□	□	□	□		□	□	□	□	□	□	
18		**Poor quality of materials provided**		□	□						□	□				□	□				
19		**shortage of resources in the market**	□	□	□	□		□	□		□	□	□		□	□	□	□	□	□	□
20		**Delay of materials delivery or equipment transportation**	□	□	□	□		□	□	□		□	□		□	□	□		□	□	
21		**Lack of availability of or equipment's failure at the site**		□	□	□			□		□	□	□		□	□	□		□	□	
22		**Lack of high Technology /mechanical equipment /software**		□	□		□			□							□				
23		**Low productivity of resources inside the country**		□	□	□	□			□	□		□		□	□	□		□	□	□
24	**Consultant related**	**Frequent changes in design from the owner**	□	□	□	□		□	□	□			□		□	□	□			□	□
25		**Mistakes and errors in design**	□	□		□			□			□	□			□	□		□		
26		**Conflicts between consultants and design engineers**		□					□	□					□	□		□			
27		**Poor/Lack of monitoring and control**	□			□						□	□	□				□			
28		**Lack of consultant's site staff Experience (managerial and supervisory personnel)**							□	□					□						
29		**Inexperienced Consultant /Project team**		□	□	□			□			□		□		□			□	□	
30		**Inflexibility of consultants**			□					□											□
31		**Poor communication & coordination between the construction parties at site**		□		□	□		□	□	□	□	□	□		□	□	□	□	□	□

32		**delay in Preparation & approval of shop drawings & material samples**		□		□			□	□		□	□		□	□		□	□	□	
33		**Lack of experience in similar projects (lack of experience)**					□					□	□				□			□	
34		**High quality of work required**	□										□				□		□		□
36		**Increase of materials and equipment cost /Fluctuation of prices/ Market inflation**		□	□	□		□	□	□	□	□	□	□	□	□	□	□	□		□
38	**Client Related**	**Delays in payment by client to contractor / subcontractor/Suppliers**	□	□	□	□			□	□	□	□	□		□	□	□	□	□	□	
41		**financial problems faced by owner**	□	□	□	□		□	□	□		□	□		□					□	
42		**Poor financial management & control on site**				□			□	□	□		□	□		□	□			□	
43		**Slowness of decision-making process (Client & Consultant)**	□	□	□	□		□	□	□	□	□	□		□	□			□	□	□
44		**Executive bureaucracy in the client's organization**							□			□					□				
45		**Suspension of work by owner**		□						□						□	□		□		
46		**Delay in revision and providing approvals for variations in design**	□	□					□	□					□	□	□	□	□		□
47		**Late handover of site/Delay to furnish and deliver the site to the contractor**		□					□			□				□			□		
48		**Change orders by owner during construction**		□	□				□	□		□		□	□	□		□	□		
49		**Inadequate/ Unrealistic estimation project time duration**	□	□	□	□	□	□	□			□		□	□	□	□		□	□	
50		**Poor security and Unprofitable delay penalties**		□								□				□	□		□		
51		**Client excessive interrfrence in project**	□					□	□			□				□	□		□		
52		**Delay in approving changes in scope of project/Contract modifiication**	□			□		□	□	□			□				□			□	□
53		**Owner has no priority/urgency to complete the project, lack of commitment,**										□					□				□
54		**Communication between the client and the contractor**		□					□				□								
55		**Lack of client's construction experience**	□	□	□			□	□			□		□		□					□
56	**Site Management Related**	**Labour strikes (other strikes)**		□						□			□		□		□		□		
57		**Complexity of project design**					□					□				□	□				
58		**Whole responsibility of site on one site engineer**											□				□				□
59		**Contractual claims, such as, extension of time with cost claims**				□						□	□		□						
60		**Accidents during construction**		□								□			□		□		□		
61		**Unavailability of HRM (human resource management)**	□	□									□		□		□				
62		**Inefficient Pre-Qualification procedures/Procurement methods used**	□				□		□	□		□			□		□	□			
63		**Absenteeism at site of consultant/Contractor/labour**				□				□			□		□		□				
64		**Improper scheduling resulting in poor judgment of time and resources**	□										□	□			□				
65		**Delay in periodically information flow among main stockholder**	□		□	□						□	□			□	□	□		□	□
66		**Insufficient project information, design and contract documents at the time of tender**		□	□	□				□		□	□	□	□	□			□		
67		**Poor documentation and no detailed written procedure**		□									□			□	□				
68		**Shortage of technical staff at construction site (equipment-operators skill)**		□	□	□	□					□	□				□	□		□	
69		**Poor contract management,**	□	□	□			□				□		□	□	□	□	□	□	□	
70		**Difficult Project Location**											□				□				□

71		**lack of clarity in scope and specification**		☐								☐	☐	☐			☐				☐
72	**External Related**	**Unfavourable\Unpredictable weather conditions**	☐	☐	☐		☐		☐	☐		☐	☐		☐	☐	☐	☐	☐		
73		**Poor and Uncertainty in (site) ground condition**	☐		☐		☐	☐	☐			☐		☐	☐		☐		☐		☐
74		**Natural effects during construction work**						☐				☐							☐		
75		**Poor government judicial system for construction dispute settlement**		☐								☐									
76		**Inappropriate government laws /regulations**		☐								☐				☐	☐	☐	☐		
77		**Delay in Government authorities' approvals/permits related to project**		☐	☐				☐			☐		☐	☐		☐		☐		
78		**Prolonged procedures of inspections and tests by consultants**		☐	☐		☐				☐	☐					☐		☐		
79		**Mistakes in soil investigation**		☐								☐				☐					
80		**Delay in resolving disputes**		☐				☐				☐		☐		☐			☐		

Above cited literature is in context of global construction industry. There are very few studies related to dam projects in Pakistan, Further studies are very rare specifically to performance evaluation of dam projects in Pakistan. Therefore, in this study, above mentioned factors are further explored in context of schedule performance in dam projects of Sindh.

3. Research objective and Methodology

Completion of every project in construction industry faces a lot of challenges. These challenges depended on some governing causes. So, this research paper is based on objective to identify the causes which effect the schedule performance of Dam construction projects on Sindh. To achieve the objective of research, firstly a comprehensive literature review was cried out and then the identified causes were delineated in tabular form to develop mapping. Afterwards, causes were divided into groups of Contractor, Consultant, Client, Project Site Management and External Factors. Furthermore, relevance and importance of causes has been explored with the help of unstructured interviews with construction industries experts having experience of dam construction projects.

4. Data Collection, Results and Discussion

Causes identified from literature were taken for further exploration in context of dam construction projects of Sindh. Industry experts were asked about the relevance and significance of causes with respect to ongoing dam projects in province through unstructured interviews. Information about respondents are shown in table

Table 4.1: Information regarding interviewed site holders.

	Position	Type of Organization	Education	Experience (Years)
1.	Project Engineer	Client	M.E (Irrigation)	15
2.	Project Manager	Client	BE (Civil Engineering)	21
3.	Project Manager	Contractor	B.E (Civil Engineering)	> 25years
4.	Assistant Resident	Consultant	Master (Management)	> 25 years
5.	Project Director	Consultant	Master (Management)	20 years

Above table shows the respondents have considerable years of experience, few of them are dealing with on-going construction of dams and they are posted on an executive designation. The respondents gave good response during interview also gave some suggestions to improve the list of causes, by indicating relevancy level. After finalizing the interviews, the collected data was statistically analysed through SPSS (Statistical Package for Social Sciences).
According to the range of adopted scale Rogers classification based on 5 points Likert scale the top ten causes were selected which were in the value range of Highly relevant (3.1-4.0) and Extremely relevant (4.1-5-0) to the schedule performance of dam construction projects. Most persistent causes explored in the research are presented in Table 4.1.

Table: 4 .2 Significant Causes Influencing Schedule Performance

S. No.	SIGNIFICANT CAUSES INFLUENCING SCHEDULE PERFORMANCE IN DAM CONSTRUCTION PROJECTS OF SINDH	N	Mean
		Statistic	Statistic
1	Financial problems faced by client	5	4.6
2	Delay in government authorities' approvals/permits related to project	5	4.4
3	Lateness of decision-making process (Client & Consultant)	5	4.4
4	Delays in payment by client to contractor	5	4.2
5	Suspension of work	5	4.2
6	Contractor insolvency or financial difficulties in case of cash flow	5	4
7	Improper planning and scheduling	5	4
8	Delays in payment to subcontractors/ suppliers	5	4
9	Shortage of skill laborers	5	4
10	Poor communication & coordination between the construction parties at site	5	4

Top five causes and their significance is discussed below:

4.1. Financial problems by client

Its most important reason in affecting schedule performance. As developing countries face budgetary issues, sometimes allocated funds could not be transferred to relevant department, consequently the projects schedule performance suffers.

4.2. Delay in government authorities' approvals/permits related to project

Any new works or variation in works need approval or permits from different government department. Permits form government departments sometimes take more time than anticipated. This might be due to lengthy and complex procedures in the relevant departments. Simplified and fast procedures will help in overcoming delays caused due to above mentioned reason.

4.3. Delay in decision making (client/consultant)

Decision making is integral part of successful project. Sometimes there are complex situation which lead towards delay in decisions. Dam projects are comparatively of longer durations and more complex from construction point of view. Extended decision-making time frames by client will create difficult situation for construction stakeholders. Therefore, mechanism of timely decision making with proper stakeholder involvement is required in client's organization.

4.4. Delay in payment by client

Delay in payment to contractor are caused sometimes due to lengthy scrutinizing procedures of client/consultant. Another main reason could be poor quality performance of contractors. Assurance of timely payments to contractors will help in overcoming schedule performance.

4.5. Suspension of work

Suspension of work may be caused due to number of reasons. One of the important reasons is the interests of main stakeholders and end user of the project. Proper stakeholder involvement at required stages of project is necessary to assure smooth work flow of construction projects.

5. Conclusion and Recommendation

Schedule performance is one of the important issues in dam construction projects. In context of dam projects in Sindh, this study has explored main causes influencing schedule performance. Few of the main causes identified in this study are: Financial problems faced by Client, Delay in government authority's approval/permits related to project, Lateness of decision-making process by client/consultant, Delay in payment by client and Suspension of work. Based on the findings of the study, following recommendations are made to improve schedule performance in dam construction projects of Sindh:

1. Due and timely transfer of allocated funds to relevant departments.
2. Simplified and fast procedures are necessary to expedite the process of required approvals/permits.
3. Mechanism of timely decision making with proper stakeholder involvement is required in client's organization.
4. Timely payments of running bills to contractor is necessary to ensure smooth work flow of project.
5. Proper stakeholder involvement at required stages of project is necessary to avoid unnecessary suspensions of work.

The finding of this well help concerned industry professional in improving schedule performance of dam projects. Moreover, similar measures can be adopted for regions having similar scenario of construction practices.

6. References

[1] https://www.opfblog.com/34145/economic-survey-of-pakistan-2018-2019-highlights/ Visited on (16-10-2019)

[2] Memon, N.A *et al*., Akram, M., Ali, T.H. &Siddiqui, F., (2015-June) Developing an Expert System for Controlling Cost and Time Performance (ESCCTO) in Construction Projects. *7 th International Civil Engineering Congress (ICEC-2015) "Sustainable Development through Advancements in Civil Engineering*", *Karachi, Pakistan.*

[3] Vaardini, S *et al*., Karthiyayini, S. and Ezhilmathi, P., 2016. Study on cost performances in

[4] VAARDINI, U.S., (2015). Identification of Causes and Impacts of Time Performance In Construction Projects. *International Journal of Applied Engineering Research*, *10*(19), p.2015.

[5] https://tribune.com.pk/story/1540651/1-1000-projects-face-delays-cost-overruns-pac/(Published in express tribune Pakistan, October 25, 2017) Visited on (15-10-2019)

[6] http://www.pbs.gov.pk/content/provisional-summary-results-6th-population-and-housing-census-2017-0 .Visited on (18-10-2019)

[7] http://finance.gov.pk/survey/chapters_19/Economic_Survey_2018_19.pdf

Visited on (20-10-2019)
[8] https://pnd.sindh.gov.pk/adp-2019-20 (IRRIGATION DEPARTMENT ANNUAL DEVELOPMENT PROGRAMME 2019-20) Visited on (22-10-2019)
[9] https://www.dawn.com/news/1269913 (*Published in Dawn, Business & Finance weekly, July 11th, 2016)* Visited on (23-10-2019)
[10] Memon, N.A *et al*., Akram, M., Ali, T.H. &Memon, A.H., (2016) Causative Factors of Time Performance in Construction Projects of Sindh. *researchgate.net*
[11] Rahman, I. A *et al*., Ismail, I. B., & Jaber, M. M. (2018). Risk Assessment of Time and Cost Performance Factors throughout Construction Project Lifecycle: Pilot Study. *International Journal of Engineering & Technology*, *7*(3.20), 717-723.**1,2**
[12] Ullah, K *et al*., Abdullah, A. H., Nagapan, S., Suhoo, S., & Khan, M. S. (2017, November). Theoretical framework of the causes of construction time and cost performances. In *IOP Conference Series: Materials Science and Engineering* (Vol. 271, No. 1, p. 012032). IOP Publishing.
[13] Shibnai, A *et al*., & Salah, K. (2015). Time and Cost Performance in Construction Projects in Egypt. *Research Gate*.
[14] Karami, H *et al*. & Olatunji, O., (2018). Delay causations in marine infrastructures, a review. In *Proceeding of International Conference on Durability of Building and Infrastructures* (pp. 167-170).
[15] Soomro, F.A *et al*., Memon, M.J., Chandio, A.F., Sohu, S. & Soomro, R., (2019). Causes of Time Performance in Construction of Building Projects in Pakistan. *Engineering, Technology & Applied Science Research*, *9*(1), pp.3762-3764.
[16] Sohu, S *et al*., Halid, A., Nagapan, S., Fattah, A., Latif, I., & Ullah, K. (2017, November). Causative factors of cost performance in highway projects of Sindh province of Pakistan. In *IOP Conference Series: Materials Science and Engineering* (Vol. 271, No. 1, p. 012036). IOP Publishing.

11th International Civil Engineering Conference (ICEC-2020) "Integrating Innovation & Sustainability in Civil Engineering" March 13-14, 2020, Karachi, Pakistan.

Use of BIM Tools In Highway Transportation System In Developed Countries - A Review

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Abstract

In recent years, there has been a trend to implement building information modelling (BIM) tools in the construction industry, keeping in mind its key advantages and barriers. Highway transportation system (HTS) is the key source of economic backbone towards the advancement. However, very little has been done about implementation of BIM tools in HTS. The purpose of this paper is to review the extent of BIM tools usage in highway transportation system in developed countries. First, the compatible BIM tools for HTS are discussed. Then, guidelines developed by few countries for use of BIM tools in HTS of developed countries are briefed along with benefits of utilizing BIM in HTS. Accordingly, hindrances in adopting BIM tools in HTS of developing countries are reviewed and possible remedies to the problems are recommended. It is recognized that BIM tools address every construction phase in HTS of developed countries, ultimately resulting in quality enhancement of the end product along with significant key advantages like saving of time and cost. And the barriers to implementation of BIM tools in developing countries are mainly lack of training and strong resistance of cultural change i.e. modern tool usage.

Keywords

Highway Transportation System, BIM, Benefits, Barriers

1. Introduction

During the years, numerous characterizations of BIM have been proposed by academics and organizations highlighting benefits like conception, development, and approximation. BIM is defined as a collection of tools which are made in order to address the concerns the problems of AEC industry by helping them to manage the construction projects by upgrading the processes of planning, design and construction (Latiffi et al. 2013). For the companies which act as designer, contractor and executors it is an analytical tool which makes it easy to store data and at the same time it can be shared with others. This way of commencement has improved the communication and coordination to improve the design and sequences of construction among the multi-disciplined teams (Sibert and Volm 2013). BIM can be also be defined as common language platform where it makes all the project multi teams into an integrated form of group. BIM main features can be distributed into sub components like detection of clashes in project, improved constructability, communication and collaborative tasks and in the end, time saving tool and easy estimate of cost (Rokooei 2015). Engineers benefit from the BIM in the form of prediction of the performance of the construction projects in the initial process which leads to tackle the fast design changes, to be able to simulate the process which in turns make a better visualization. This whole BIM process outputs a construction documentation which is up to a better quality and a standardize format. This post analysis process is valuable for the design team to earlier extraction of data which helps them to make the project economical (Strafaci 2008).

BIM has been seen by many industrialists and academicians as a growing trend in the construction industry. BIM provides a disciplined and rationalized tactic for building developments but its application for highway projects are infrequent but conventional approaches and technologies of BIM demonstrate vast potential in advancing the highway transportation industry. There has been seen a necessity to mature up and employ advanced technologies in the transportation division to lift up the country's economy and provide comfortable journey for the travelers. BIM for the application of highway scan be transformed into Highway information modelling (HIM) which necessitates design, quantity take-off, simulation, clash detection and other benefiting uses (Ertaymaz and Atasoy2019). BIM for highway transportation system is also used in the form of bridge information modelling (BrIM). Building information modeling (BIM) is a new technology in bridge construction industry. 3D models can provide perfect numerical expression of drawings from design results. 3D information models for bridge structures improve design quality in terms of accurate drawings, constructability and collaboration (Shim et al. 2011). Building information modeling links and analyzes data related to the inspection, evaluation, and management of bridges. BIM facilitates the inspection and evaluation of bridges, which enables transportation companies to proficiently manage bridge inventories and lead to a more mechanized practice (McGuire et al. 2016). This paper explores the range of BIM tools usage in HTS of developed countries with addition of highlighting the potential benefits and hinderances to their adoption. Meanwhile, guidelines by various countries on BIM for highway will also be mentioned to highlight the main attributes by which BIM is inculcated to the industrial process.

2. Highway Transportation System

Transportation system is defined as combination of vehicles, road for guidance and plan which commences operation to move people and goods (Ran and Boyce 2012). In another definition, Highway transportation system is defined as a web of nodes which align themselves against arcs on which vehicles are bound to move whereas the nodes are described as cities, intersections (Durbin 1966). Transportation system is a critical fragment of civic infrastructure effecting economy and have massive influence on the profile of the society and the competence of the economy in overall. Transportation system is denoted as a composite system of roads and highways, railroads, airports, waterways, and inner-city transportation systems which provides the movement of public and possessions. Transportation system addresses safety, economy and traffic issues. Transportation developers and engineers put effort to provide bulk space for detected or predicted travel mandate by construction of resourceful transportation systems producing both flexibility and convenience. The governments of every developed country deliver specific large amount of capital for the construction, preservation and maneuver of highway and other transportation systems (Roess et al. 2004).

Among the developing countries, U.S highway transportation system is observed as leading example but the transportation system of U.S still faces problems with maintenance and management issues. Other countries, can follow the example but there is a need of study of how the system got developed (Boarnet 2014). Some of the challenges that are being faced by the developing countries are mentioned. Case study on China's highway transportation system, reveals that there was excessive feeding of energy which lead to inefficient environmental index (Song et al. 2016). Also, when talking about the construction there are also issues in the construction phases too like there are problems regarding inappropriate site management, no updated software and hardware equipment for construction of highways and the fast-changing design changes this leads to effect on time, project cost and low-quality production (Santoso and Soeng 2016). Although, BIM could be applied for every civil engineering element, but recent study in Taiwan shows that for the road infrastructure it is not being used at an abundant value although participant in the questionnaire for BIM answer efficient use for road infrastructure is very helpful (Chang and Lin 2014).

3. BIM Tools and Usage in Highways

The BIM tools which are mainly incorporated for the purpose of design, construction and maintenance of transportation projects are: Auto CAD Map 3D, Storm and Sanitary Analysis, Infra works, Auto

CAD Civil 3D, Bridge module, Rail Layout module, River and flood analysis module, Robot structural analysis professional (Chong et al. 2016). An example of BIM used in highway is shown below. The main software for BIM that are provided by companies like in the UK, Neil Brazier Ltd. Company provides the software like Auto Desk Civil 3D, Navisworks, Infra works and Revit. According to the company, these BIM services provide control of the project's life cycle and a better pre view of project.

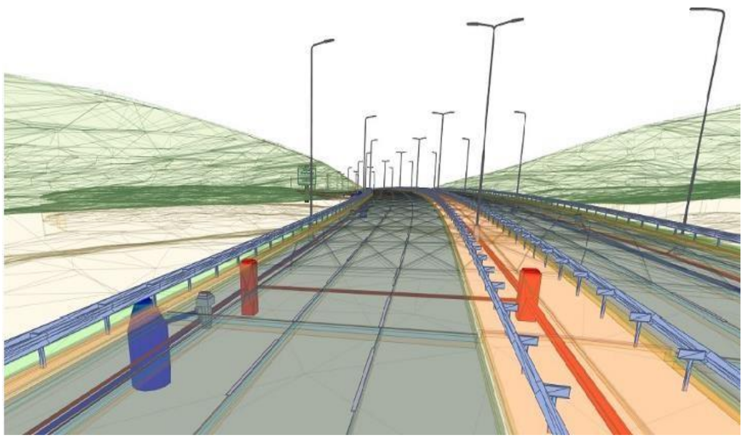

Figure 1: BIM Used in Highway (Neil Brazier Ltd.)

Some of the major specialists related to BIM are: BIM developer like draftsperson; BIM investigator for analysis; BIM developer for the software; BIM coordinator for assisting the non-BIM users; BIM experts secondly named as consultants for providing software and assisting the companies which are new to BIM implementation; BIM educators and scientists for researcher purposes; BIM administrator or manager or BIM project manager for training of employees, coordination and implementation of BIM software (Barison and Santos 2010). Table 1 explains the uses of BIM in a highway project from the pre-construction to construction and post construction phases (Chong et al. 2016).

Table 1: BIM Uses for Highway Implementation Phase (Chong et al.)

	Highway Construction Phase		
	Pre-Construction	**Construction**	**Post-Construction**
BIM Uses	a) Setting out the site b) Drawing layout of site c) Coordination d) Project scheduling e) Management of material f) Engineering analysis	a) Construction inspection b) Human resource and progress tracking c) Quality assurance d) Safety onsite e) Cost control f) Constructability reviews	a) Planned maintenance b) System analysis c) Asset management d) Emergency plan e) Transportation management

4. BIM International Guidelines in Highways

At first BIM guidelines for buildings will be reviewed. The book "Handbook A Guide to Building Information Modeling for Owners, Managers, Designers, Engineers, and Contractors" is an absolute example as guidelines which act as a learning guide to implement BIM in the construction industry. This guideline delivers an awareness of BIM technologies, issues related with BIM implementation, and the effects by use of BIM provides to a project team. The guideline consists of introduction to BIM and technologies and also describing the potential benefits of BIM. The guideline also mentions perceptions of BIM related to specific disciplines like addressing to owners and facility managers, architects and engineers, construction industry and subcontractors and fabricators. The guideline to assist in the procedure of BIM also presents case studies in which BIM is used and mentions the experiences and practices of owners, architects, engineers, contractors and fabricators. Furthermore, this guideline also puts ahead a futuristic vison for the coming years and also mentioning the current trends which are being employed; this helps in setting out a direction for the future use for BIM (Eastman et al. 2011)

Queensland government has published a guideline for the road projects for the implementation of BIM which is an outcome for the policy on digitalizing the engineering field. The guideline outlines BIM approaches to deliver a road project by implementing BIM. The guide first introduces the BIM to the readers where its definitions, uses and benefits for the transportation are mentioned. Along with the BIM definitions, the guideline also clears the wave of uncertainty of legal applications by mentioning the particular responsibilities and rules for application for both the consultant and contractor. The guidelines also mention what are the deliverables of a project which implements BIM. Furthermore, the guidelines mention the practices to be followed for BIM modeling and documentation details.

5. BIM Benefits for Highway

There are many studies related to BIM advantages. Before progressing towards the BIM benefits for highway, firstly there will be a mention for BIM advantages in overall construction industry to judge the similarity between the BIM advantages in highway and overall construction industry. The most frequent benefit of BIM is time and cost savings which are most important factors of a project dependency (Bryde 2013). According to the study done by Bimal Kumar (2017), who observed a case study in which BIM was implemented in an infrastructure project in Scotland. The cases study shows that two cases were chosen were chosen for the project. This project included both BIM software and the CAD which is traditionally used all around the world. This study done in parallel to BIM and CAD outlined the benefits of implementing the BIM. The end results showed that by implementing BIM there was significant increase in the savings of the project. This happened due to common information exchange among stake holders on the platform given by BIM. Other benefits show that there was improvement in coordination among project members, the clash detections were easy and faster at the early stages and the efficiency of the project increased subsequently. Study by Ben Sibert et al. (2013) outlines the advantages of using BIM in highway by going through a practical case study. The key advantages of BIM assessed are that BIM at its initial plan phases covered up all the work stages of the construction including the design phases. The model was created which comprised of features that were above the ground level. This model was then processed into a video which is a unique capability of BIM. This video was developed in the draft stages that communicated the public and the client. This process led to earlier detection of problems in the model by the help of input by public and major stake holders. This is unique from 2D drawings as it shows better visualization of the project and understanding to all

One of the studies which implements BIM on the road shows that the key advantages shown by BIM are that there is less involvement of human participation in the road project which in turns reduces the labor cost. There are increased chances of fast and accurate work, less consumption of materials on the grading process and also less passes to be laid on the road. There are more night and safe works at the construction process of road (Znobishchev and Shamraeva 2019). When BIM and GIS were utilized in a national road in the feasibility phase it showed advantages as time saving, cost reduction, better 3 D visualization of project and the flexibility to change the data according to routes if there is a need of design change (Park 2014). Case study on implementation on Bridge construction also signifies savings in the financial aspect of project by 5-9% following the utilization of BIM (Fanning et al. 2014). Table 2 shows some of the benefits of BIM for overall construction industry.

Table 2: BIM benefits for Construction industry

Serial No.	Author	BIM benefits
1	Yan and Demian (2008)	a) Saves the design cost b) Reduces human resource c) Digitalizes the process of design and build
2	Migilinskas et al. (2013)	a) Time saving by reduction of corrections b) Reduction of Human Nature Mistakes c) Saving cost by accurate bill of quantities
3	Blanco and Chen (2014)	a) Improved Co ordination b) Efficient process leading to less waste c) Earlier clash detection d) Less chance of redoing of work e) Saves the time f) Makes the project economical
4	Ozorhon and Karahan (2016)	a) Qualified Staff b) Enhanced Technology and updated information
4	Jin (2017)	a) Subsequent decrement of design errors b) No chance of rework
5	Chan et al. (2019)	a) better cost estimation b) efficient construction planning and management c) improvement in design and project quality.

6. Hinderance In Adopting BIM In Highway for Developed Countries

Likewise, just in the case of advantages of BIM here first hinderances of BIM implementation related to construction industry will be explained then afterwards the BIM barriers in the highway will be explained to explore similarity between them. In the construction industry, some of the barriers that are linked to BIM implementation are that the users undergo a thinking that the companies do not opt to BIM because for this process company has to put a lot of time in the training process which also increases the human resources. While the architects do not want to go for a change because they are contented with the current methods like CAD. They are also doubtful that they the newly developed function will have any advantages (Yan and Demian 2008). Another study shows that there is deficiency of the support from the senior administration. There is a lot of cost for the implementation of the BIM software. And in the end, there are legal issues while using BIM in the projects (Eadie et al. 2014). One of the greatest threats to BIM implementation are the people and organizations, like

clients which are the main key to implementation but they are of a tough that BIM is not ready to be accepted in market and pose BIM a threat for increasing the project's cost (Porwal and Hewage 2013). Some other barriers reported were linked to four metrices; organization issues, technicality, financial aspects, contractual issues and legal issues. The problems faced were similar to the studies as above (Alreshidi et al. 2017).
In the highway, study by Shabaan Khalid et al. (2015) shows that the main hinderances in the BIM implementation are there will be need for extra investment for in BIM software and hardware, no training platforms provided for awareness of BIM, the practitioners want to stick to the traditional methods resisting for any change and this whole idea of BIM will lead to extra burden of work.

8. Conclusion

In this paper, extent of BIM tools usage in highway transportation system in developed countries was discussed. The paper reviewed the compatible BIM tools uses along with the befits and their adoption hinderances. There was also mention of guidelines for transportation. By reviewing the studies, the main thing observed was there has been many studies of BIM tools usage in buildings but few studies have been done for the highways and the benefits that have been significant in building have the same similarity as of highways. The papers reviewed for the BIM tools usage shows that mainly the tools which are available for the highways are Auto Desk Civil 3D, Navisworks, infra works and Revit which are the products of AutoCAD. These tools are used in the pre-construction, construction and post construction phases of highway. While the benefits of using BIM tools are manly saving of time, cost, better design of highway roads and improved quality projects which were mainly observed in the review of papers. Government of Queensland have published a guideline for the implementation of BIM in highways by highlighting BIM uses in transport, BIM benefits for the highways and the procedures to follow BIM in the highway project execution. As the study shows many significant advantages of BIM tools but there are hurdles for the implementation of BIM on highways like there is misconception of extra investment to be done for BIM, practitioners want to stick on to the traditional methods and mostly it will lead to extra work. The whole study concludes that there must be emphasis of BIM tools implementation on highways as it has similar benefits like as in the buildings and the hinderances to the implementation must be addressed by publishing guidelines highlighting the benefits of BIM tools in highway transportation system.

9. Acknowledgement

The authors would like to thank every person who helped in the study, especially Huzayfa Ghazi and Muhammad Darwaish.

References

Alreshidi, E., Mourshed, M., & Rezgui, Y. (2017). Factors for effective BIM governance. Journal of Building Engineering, 10, 89-101.
Barison, M. B., & Santos, E. T. (2010, June). An overview of BIM specialists. In Proceedings of the International Conference on Computing in Civil and Building Engineering (pp. 141-146). Nottingham University Press Nottingham, UK.
Blanco, F. G. B., & Chen, H. (2014). The implementation of building information modelling in the United Kingdom by the transport industry. Procedia-Social and Behavioral Sciences, 138, 510-520.
Boarnet, M. G. (2014). National transportation planning: Lessons from the US Interstate Highways. Transport Policy, 31, 73-82.
Bryde, D., Broquetas, M., & Volm, J. M. (2013). The project benefits of building information modelling (BIM). International journal of project management, 31(7), 971-980. (2)

Chan, D. W., Olawumi, T. O., & Ho, A. M. (2019). Perceived benefits of and barriers to Building Information Modelling (BIM) implementation in construction: The case of Hong Kong. *Journal of Building Engineering*

Chang, J. R., & Lin, H. S. (2014). Underground pipeline management based on road information modeling to assist in road management. *Journal of Performance of Constructed Facilities*, *30*(1), C4014001.

Chong, H. Y., Lopez, R., Wang, J., Wang, X., & Zhao, Z. (2016). Comparative analysis on the adoption and use of BIM in road infrastructure projects. *Journal of Management in Engineering*, 32(6), 05016021.

Durbin, E. P. (1966). An interdiction model of highway transportation (No. RM-4945-PR). RAND CORP SANTA MONICA CA.

Eadie, R., Odeyinka, H., Browne, M., McKeown, C., & Yohanis, M. (2014). Building information modelling adoption: an analysis of the barriers to implementation. Journal of Engineering and Architecture, 2(1), 77- 101.

Eastman, C. M., Eastman, C., Teicholz, P., Sacks, R., & Liston, K. (2011). *BIM handbook: A guide to building information modeling for owners, managers, designers, engineers and contractors*. John Wiley & Sons.

Ertaymaz, U., & Atasoy, G. (2019). Modeling highway projects: The need for Highway Information Modeling (HIM) guideline and information exchange. *Journal of Construction Engineering*, *2*(1), 10-17.

Fanning, B., Clevenger, C. M., Ozbek, M. E., & Mahmoud, H. (2014). Implementing BIM on infrastructure: Comparison of two bridge construction projects. *Practice Periodical on Structural Design and Construction*, *20*(4), 04014044.

Jin, R., Hancock, C., Tang, L., Chen, C., Wanatowski, D., & Yang, L. (2017). Empirical study of BIM implementation–based perceptions among Chinese practitioners. *Journal of Management in Engineering*, *33*(5), 04017025.

Kumar, B., Cai, H., & Hastak, M. (2017). An assessment of benefits of using BIM on an infrastructure project. In *International Conference on Sustainable Infrastructure* 2017 (pp. 88-95).

Latiffi, A. A., Mohd, S., Kasim, N., & Fathi, M. S. (2013). Building information modeling (BIM) application in Malaysian construction industry. *International Journal of Construction Engineering and Management*, 2(4A), 1-6.

McGuire, B., Atadero, R., Clevenger, C., & Ozbek, M. (2016). Bridge information modeling for inspection and evaluation. *Journal of Bridge Engineering*, *21*(4), 04015076.

Migilinskas, D., Popov, V., Juocevicius, V., & Ustinovichius, L. (2013). The benefits, obstacles and problems of practical BIM implementation. *Procedia Engineering*, *57*, 767-774.

Ozorhon, B., & Karahan, U. (2016). Critical success factors of building information modeling implementation. *Journal of Management in Engineering*, *33*(3), 04016054.

Park, T., Kang, T., Lee, Y., & Seo, K. (2014). Project cost estimation of national road in preliminary feasibility stage using BIM/GIS platform. In *Computing in Civil and Building Engineering (2014)* (pp. 423-430).

Porwal, A., & Hewage, K. N. (2013). Building Information Modeling (BIM) partnering framework for public construction projects. *Automation in construction*, *31*, 204-214.

Queensland Government Building Information Modeling (2017, May). Retrived from https://www.tmr.qld.gov.au/business-industry/Technical-standards-publications/Building-Information Modelling.

Ran, B., & Boyce, D. (2012). Dynamic urban transportation network models: theory and implications for intelligent vehicle-highway systems (Vol. 417). Springer Science & Business Media

Roess, R. P., Prassas, E. S., & McShane, W. R. (2004). *Traffic engineering*. Pearson/Prentice Hall.

Rokooei, S. (2015). Building information modeling in project management: necessities, challenges and outcomes. *Procedia-Social and Behavioral Sciences*, 210, 87-95.

Santoso, D. S., & Soeng, S. (2016). Analyzing delays of road construction projects in Cambodia: Causes and effects. *Journal of Management in Engineering*, *32*(6), 05016020.

Shaaban, K., & Nadeem, A. (2015). Professionals' perception towards using building information modelling (BIM) in the highway and infrastructure projects. *International Journal of Engineering Management and Economics*, 5(3-4), 273-289.

Shim, C. S., Yun, N. R., & Song, H. H. (2011). Application of 3D bridge information modeling to design and construction of bridges. *Procedia Engineering*

Sibert, B. (2013, February). Using building information modeling on a highway project. In Proceedings of the Institution of Civil Engineers-Civil Engineering (Vol. 166, No. 1, pp. 9-9). Thomas Telford Ltd.

Song, M., Zheng, W., & Wang, Z. (2016). Environmental efficiency and energy consumption of highway transportation systems in China. *International Journal of Production Economics*, *181*, 441-449.

Strafaci, A. (2008). What does BIM mean for Civil Engineers. CE News, Tranportation.

Yan, H., & Demian, P. (2008). Benefits and barriers of building information modelling. IN: Ren, A., Ma, Z. and Lu, X. In *Proceedings of the 12th International Conference on Computing in Civil and Building Engineering (ICCCBE XII) & 2008 International Conference on Information Technology in Construction (INCITE 2008), Beijing, China, 16th-18th October*. Tingshua University Press.

Znobishchev, S., & Shamraeva, V. (2019). Practical Use Of Bim Modeling For Road Infrastructure Facilities. Architecture and Engineering, 4(3), 49-54.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Sustainable Construction in Balochistan: Issues and Challenges

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Abstract

Pakistan being an underdeveloped country faces endless crises such as lack of energy resources, deforestation and water pollution. These problems have plagued the country for decades and are likely to persist in the future. Balochistan being the most underdeveloped part of Pakistan is engulfed by many other issues like lack of rainfall, mismanagement of rain water, shortage of electricity, excessive mining of vulnerable natural resources and destruction of natural habitat are impediments to sustainable construction. If this continues, a time may come when construction would not be sustainable anymore. To avoid such a possibility, this research aims to identify the impediments to sustainable construction in Balochistan. The results of this study shows that lack of professional knowledge, lack of demand for green buildings, lack of building codes & regulations, lack of training in sustainable design/construction and lack of government support are the most significant barriers to green building practices. The findings of this study help to understand the major barriers to sustainable construction and aid the policy makers to take measures and informed decisions regarding green building practices in Balochistan.

Keywords: Sustainable Construction, Green Buildings, Barriers, Construction Industry

1. Introduction

The change in climate has caused unvarying increase in temperature all over the world, mainly in South Asia. Summer duration has extremely increased. Besides the rainfall has reduced in Mediterranean and Southern Asia (Flying, 2010). Sustainable construction is a continuous state of development defined as: The practice of rising the efficiency with which structures and their sites use materials, energy and water and decrease impact of structures on the environment and on the human health, by better design, citing, construction, operation and maintenance (Azad and Akbar, 2010). Similarly, the Environmental Protection Agency (EPA, 2010) defines sustainable design as "The exercise of building structures and utilizing processes that is resource-efficient and environmentally responsible throughout a structures life cycle from conception to design, construction, operation and maintence (Baum and Morgan, 2007). Green building is an example of high performance or sustainable building (Francis and Hoban, 2002)

Currently, Pakistan is facing a number of economic and environmental challenges (Azeem, et al., 2017). It has been a sufferer of acute energy crises in the last few years (Javaid et, al., 2011). Due to hot climate of Pakistan, energy demands for cooling buildings are very high (Sohail et al., 2010). It is on the list of those countries who largely depend upon thermal resources and produce most of their electric energy from non-renewable resources (Ahmed and Iftikhar-ul-Husnain, 2014). Balochistan being the most underdeveloped part of Pakistan is engulfed by many other issues like lack of rainfall, mismanagement of rain water, shortage of electricity, excessive mining of vulnerable natural resources and destruction of natural habitat are impediments to sustainable construction. This tradition is producing severe environmental issues, alongside with fast consumption of valuable sources of energy (Aslam et al., 2012). To prevent a shortage of natural resources, there is dire need to move towards sustainable and energy-efficient building by changing the current construction practices (Zainordin et al., 2012). Sustainable building has the capacity to save up to 30 percent. Green building has the potential to reduce solid waste in construction by 70 percent, water utilization by 40 percent and carbon dioxide emissions by 39 percent (Bohari et al., 2016).

It is unfortunate that, in Balochistan the sustainable construction approach suffers from numerous market impediments, inspite of its many advantages to society. Sustainable construction is the only feasible solution and is an effectual alternate to energy-efficient structures. This research study is an effort to investigate the barriers to sustainable construction in Balochistan. Based upon the results of the study recommendations are suggested that can be productive in encouraging sustainable construction in Balochistan.

2. Research Methodology

To carry out the research study both quantitative and qualitative approaches were used. An extensive literature review was carried out by critically reviewing the journals, books and research papers. Flow chart of the methodology has been shown in figure 1. Twenty (20) barriers were identified as shown in table 1. After a pilot survey, the questionnaire was further reviewed by reducing the barriers to fifteen (15).

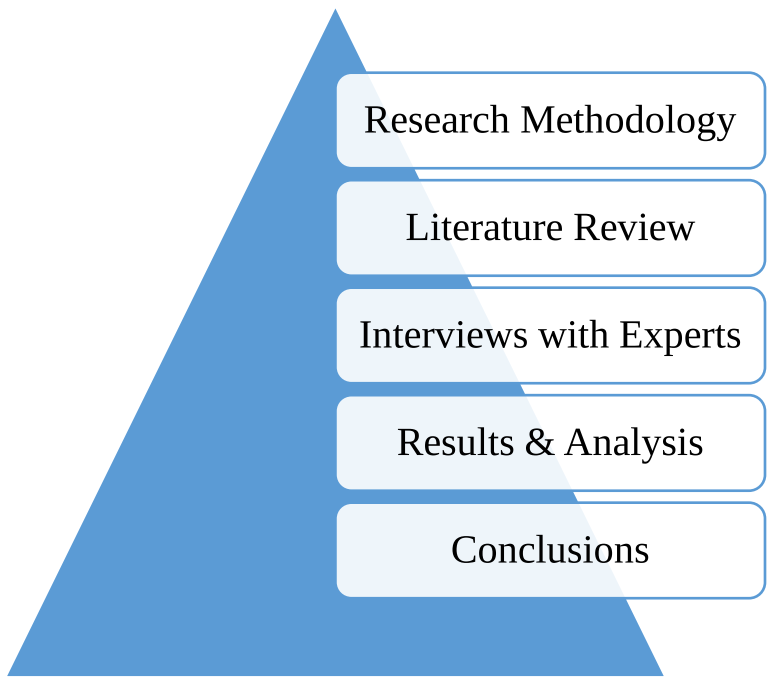

Figure 1: Research Methodology

Table: 1 Worldwide Barriers in Sustainable Construction

S.No.	Barriers	Key References
1	Lack of building codes & regulations	Bohari et al. (2016), Ali et al. (2016), Wang et al.(2016), Milad S (2013)
2	Lack of professional knowledge	Ali et al. (2016), Ametepey et al. (2015), Soheila B (2008),
3	High initial cost	Timilsina et al. (2016), Akadiri (2015), Milad S (2013), Williams and Dair (2007),
4	Lack of policy to promote green building	Azad and Akbar (2015), Persson and Grönkvist (2015), Milad S (2013), Samari et al. (2013),
5	Lack of education /training in sustainable design/construction	Ghazilla et al. (2015), Milad S (2013), Miriam L (1999),

6	Resistance to cultural change from traditional to green construction	Al Sanad (2015),Kasai and Jabbour (2014), Miriam L (1999)
7	Lack of public awareness	Timilsina et al. (2016), Attaran and Celik (2015), Milad S (2013), Soheila B (2008)
8	Lack of technology	Ali et al. (2016), Kasai and Jabbour (2014), Milad S (2013), Samari et al. (2013
9	Lack of government support	Ametepey et al. (2015), Milad S (2013), Miriam L (1999),
10	Lack of demand for green buildings	Persson and Grönkvist (2015), Ametepey et al. (2015), Milad S (2013)
11	Lack of incentives	AlSanad (2015), Azad and Akbar (2015), Samari et al. (2013), Wood (2007)
12	Lack of qualified team	Kasai and Jabbour (2014), Milad S (2013), Miriam L (1999)
13	Uncertainty in the performance of equipment and green materials	Ghazilla et al. (2015), Milad S (2013)
14	Imperfect green technology specifications	Wang et al. (2016), Samari et al. (2013), Milad S (2013)
15	Risk associated with green construction	Al Sanad (2015), Miriam L (1999),
16	Lack of top management support and time to implement green construction	Yin et al. (2018), Bohari et al. (2016), Ghazilla et al. (2015),
17	Long payback period	Attaran and Celik (2015), Akadiri (2015), Milad S (2013),Williams and Dair (2007),
18	Higher maintenance cost of green building	Milad S (2013), Soheila B (2008), Williams and Dair (2007)
19	Lack of financial resources	Ametepey et al. (2015), Ghazilla et al. (2015),), Milad S (2013
20	Unsufficient communication structure to support green construction	Ghazilla et al. (2015), Dahle and Neumayer (2001),

3. Data Collection

Interviews were conducted in order to investigate key barriers in implementation of sustainable construction in Balochistan. For this arbitrarily selected experts of the construction sector were interviewed and asked to identify the current level of effectiveness of impediments to construction industry of Baochistan. The demography of the panel participating in interviews is as in Table 2.

Table 2: Demography of Respondents

S. No	Expert	Type of Organization	Experience (years)
1	Executive Engineer	Client	21 or above
2	Additional Director	Client	21 or above
3	Executive Engineer	Client	16 – 20
4	Team Leader	Consultant	15 – 20
5	Procurement Manager	Consultant	06 – 10
6	Chief Executive	Contractor	15 – 20

7	Chief Executive	Contractor	15 – 20
8	Project Manager	Contractor	15 – 20
9	Resident Engineer	Contractor	06 – 10
10	Assistant Director	Client	06 – 10
11	Assistant Engineer	Client	03-05

From table 2 above, it is observed that all the experts have remarkable experience of working in the construction industry. They are working in leading positions in their organizations. The participants represented the major stakeholders of the construction industry which include client, contractors and consultant.

4. Results and Discussion

In order to identify the most critical barriers in adoption of sustainable construction practices in Balochistan, respondents were asked to rank the barriers mentioned in the questionnaire according to their current level of effectiveness. The results have been shown in Table 3. This shows the top ranked barriers exists towards implementation of green construction.

Table: 3 Ranking of Barriers

Barriers	Ranking by experts
Lack of professional knowledge	1
Lack of demand for green buildings	2
Lack of building codes & regulations	3
Lack of education /training in sustainable design/construction	4
Lack of government support	5
Lack of technology	6
High initial cost	7
Lack of incentives	8
Lack of financial resources	9
Lack of qualified team	10

It is observed from Table 3 that “lack of professional knowledge” is one of the most critical barrier to adoption of sustainable construction in Balochistan. “Lack of demand for green building” is ranked second and lack of building codes and regulations is ranked third. “Lack of education / training in sustainable design /construction” and “lack of government support” are ranked fourth and fifth respectively.

5. Conclusions & Recommendations

According to findings of the study construction experts believed that lack of understanding sustainability is a major obstacle to green building practices. The sustainable construction is a newer approach in construction industry of Balochistan and professionals who have sound technical knowledge and experience are limited in number. Lack of demand for green building is ranked as second most significant barrier, as the interviewees see weak market demand, lack of demand for green construction by clients and stakeholders. Lack of building codes and regulations is also one of the barriers highlighted by interviewees. The construction experts found it hard to begin a project without the help of sustainable construction guidelines. In order to adopt the green building practices the internationally recognized rating systems and sustainable building guidelines needed to be imported. The results of the study shows that education /training in green buildings plays a vital role, ranking fourth. So training should be provided and dedicated courses on sustainability must be added in curriculum of universities. The finding of this study also highlighted lack of technology, lack of incentives from government and qualified team as major barriers to sustainable construction. The results of this study help to understand the major impediments to sustainable construction and aid the policy makers to take measures and informed decisions regarding green building practices in Balochistan.

References

Ahmed, A. and Iftikhar-ul-Husnain, M, “Energy Smart Buildings: Potential for Conservation and Efficiency of Energy*”, Pakistan Development Review, Vol. 53 (2014), pp. 371-380*

Akadiri, P.O, “Understanding Barriers Affecting the Selection of Sustainable Materials in Building Projects”, *Journal of Building Engineering, Vol. 4 (2015), pp. 86-93.*

Ali, A.N.A., Jainudin, N.A., Tawie, R. and Jugah, I, “Green Initiatives in Kota Kinabalu Construction Industry”, *Procedia – Social and Behavioral Sciences, Vol. 224 (2016) pp. 626-631.*

AlSanad, S, “Awareness, Drivers, Actions, and Barriers of Sustainable Construction in Kuwait”, *Procedia Engineering, Vol. 118 (2015), pp. 969-983.*

Aslam, A., Tariq, S., Syed, W.A. and Ali, S.S, “Green Architecture & Environmental Benefits:A Review with Reference to Energy Deficient Pakistan”, *Science International Lahore, Vol. 24 (2012), pp. 495-498.*

Attaran, S. and Celik, B.G, “Students’ Environmental Responsibility and Their Willingness to Pay for Green Buildings”, *International Journal of Sustainability in Higher Education, Vol. 16 (2015), pp. 327-340.*

Azad.S, Akbar.Z, *“*The Impediments in Construction of Sustainable Buildings in Pakistan*” European Scientific Journal , Vol.11 (2015) pp.369-37.,*

Azeem.S, Naeem.M.A and Waheed.A. “Examining Barriers and Measures to Promote the Adoption of Green Building Practices in Pakistan, *Journal of Smart and Sustainable Built Environment*, Vol.6 (2017) pp-86-100.

Baum. M and Morgan. W. K “Sustainable Building is the Forthcoming Necessity” *Am. J. Public Health. , Vol.*64 *(2007) pp-212-4.*

Bohari, A.A.M., Skitmore, M., Xia, B. and Zhang, X, “Insights into the Adoption of Green Construction in Malaysia: the Drivers and Challenges”, *Environment-Behaviour Proceedings Journal, Vol. 1 (2016), pp. 45-53.*

Dahle, M. and Neumayer, E, “Overcoming Barriers To Campus Greening: A Survey Among Higher Educational Institutions in London, UK”, *International Journal of Sustainability in Higher Education, Vol. 2 (2001) pp. 139-160.*

Flying JC Howe, “The Increasing Prevalence and Severity of Summer in the Asia Due to Increasing Climate Change” *Occup. Environ. Const.Vol. 69 (2010) pp.649-50.*

Francis. H and Hoban. V. Vallyathan, “How People Perceive to Live Comfortable in Well Cited Buildings” *Occup. Environ, Vol. 62(2002) pp-207-43.*

Ghazilla, R.A.R., Sakundarini, N., Abdul-Rashid, S.H., Ayub, N.S., Olugu, E.U. and Musa, S.N. (2015),“Drivers and Barriers Analysis for Green Manufacturing Practices in Malaysian SMEs: a Preliminary Findings”, *Procedia CIRP, Vol. 26(2015), pp. 658-663.*

Javaid, A., Hussain, S., Maqsood, A., Arshad, Z., Arshad, A. and Idrees, M , "Electrical Energy Crisis in Pakistan and Their Possible Solutions", *International Journal of Basic & Applied Sciences, Vol. 11 (2011), pp. 26-35.*
Kasai, N. and Jabbour, C.J.C, "Barriers to Green Buildings at Two Brazilian Engineering Schools", *International Journal of Sustainable Built Environment, Vol. 3 (2014), pp. 87-95.*
Persson, J. and Grönkvist, S, "Drivers for and Barriers to Low-Energy Buildings in Sweden", *Journal of Cleaner Production, Vol. 109 (2015), pp. 296-304.*
Samari, M., Ghodrati, N., Esmaeilifar, R., Olfat, P. and Shafiei, M.W.M, "The investigation of the Barriers in Developing Green Building in Malaysia", *Modern Applied Science, Vol. 7(2013), pp. 1-10.*
Sohail, A. and Qureshi, M.U.D, "Energy-Efficient buildings in Pakistan", *A Scientific Journal of Comsats – Science Vision, Vol 16 (2010).*
Timilsina, G.R., Hochman, G. and Fedets, I, "Understanding Energy Efficiency Barriers in Ukraine: Insights From a Survey of Commercial and Industrial Firms", *Energy, Vol. 106 (2016) pp. 203-211.*
Wang, T., Li, X., Liao, P.-C. and Fang, D, "Building Energy Efficiency for Public Hospitals and Healthcare Facilities in China: Barriers and Drivers", Energy, Vol. 103 (2016) pp. 588-597.
Williams, K. and Dair, C, "What is Stopping Sustainable Building in England? Barriers Experienced by Stakeholders in Delivering Sustainable Developments", *Sustainable Development-Bradford, Vol. 15 (2007), pp. 135-147.*
Zainordin, N., Abdullah, S.M. and Baharum, Z, "Users Perception Towards Energy Efficient Buildings", *Asian Journal Environment Study, Vol. 3 (2012), pp. 91-105.*

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Water Saving Techniques in Civil Engineering Department Of Mehran UET Jamshoro

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Abstract

Life on the earth is impossible without water. Water which is available on the earth includes 97% of brackish water which remain in oceans and 3% of fresh water so there is a need that the use of fresh water should be in a proper way and sustainable manner which will benefice our successors. Water quantity is depleting day by day and Pakistan is considered among water stressed countries of the world. So water must be used wisely and consider it as a precious gift of nature. Everyone should take part to conserve this source of life. It is time of need to propagate awareness among the people about the importance of water. Every work starts from own self so this research work is conducted in Civil Engineering department. In this study, it is observed that most of the water is wasted due to hot water flowing in the pipe network during day time. Hence temperature plays a vital role in the loss of water. Water remains too hot due to direct exposure of conveying pipes to the sunlight. The normal temperature ranges between 15-41^{0}C throughout the day during months of March to October. This issue may be solved by covering exposed section of pipe with gunny bags. By this act one can save more than 34% water daily in particular application.

Keywords
Water Saving, Availability, Temperature Effect, Civil Engineering Department, MUET Jamshoro.

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1. Introduction

Concern for the health of the global environment has increased in recent years. A safe adequate water supply for 1.8 billion people, about one-third of the world population, is still a dream. 97% of the water on the Earth is salt water and only three percent is fresh water; slightly over two thirds of this is frozen in glaciers and polar ice caps. Water is one of the prime necessities of life. We can hardly live for a few days without water. In a man's body, 70-80% is water. Cell, blood, and bones contain 90%, 75%, and 22% water, respectively.

In 2000, the world population was 6.2 billion. The UN estimates that by 2050 there will be an additional 3.5 billion people with most of the growth in developing countries that already suffer water stress. Thus, water demand will increase unless there are corresponding increases in water conservation and recycling of this vital resource. In 2025, water shortages will be more prevalent among poorer countries where resources are limited and population growth is rapid, such as the Middle East, Africa, and parts of Asia. Pakistan will face very severe water shortages due to physical scarcity and a condition of overpopulation.

We need water to grow and to stay alive. The world's usable freshwater supply is being depleted because of increasing population and contamination. As world population grows, the demand for freshwater is increasing for household uses, industrial uses and food production. Population growth and density also affect the availability and quality of water resources in areas where people obtain their water supply. Overuse of water can lead to the depletion of surface and groundwater resources, causing water shortages. It is important for everyone to save water so that become adequate for everyone. Water conservation means using our water wisely and caring for it properly. It is our responsibility to learn more about water conservation and how we can keep our water pure and safe for generations to come so that we all can enjoy the benefits of having pure and clean water.

This study is motivated by the increasing need and to promote better water resources management in Civil Engineering Department of Mehran University Engineering and Technology Jamshoro. This need also arises in multiple other departments of MUET but the number of students in Civil Engineering Department is equal to aggregate of multiple other department similarly the account of teacher and staff is available more than others. Excess of water is lost in washroom, landscaping, gardening and drinking purpose as well. That is why we did research on Civil engineering department because if Civil Engineering Department is able to provide safe, adequate water to students and staff then the problems in other departments overwhelmed automatically.

2. Aims and Objectives

The aim of this study is to save water for future generations. The specific objectives of the study are listed below.

1. To check the physical and chemical qualities of water.
2. To calculate the quality of water being used in Civil Engineering Department.
3. To detect the issues related to losses of water.

3.Literature Review

Water Saving techniques have been studied by several engineers and scientists. Practically, all investigations aim that how to conserve the water by chemically or physically for future generations.

E. Posadas et al., 2014 investigated the effect of Phormidium, Oocystis & Microspora on the removal of nitrogen from industrial wastewater and the removal efficiency obtained 85%. **Ignacio et al., 2009** checked the effect of Chlorella sorokiniana on nutrients removal from industrial wastewater and resulted in the removal of 94% of nitrogen. A study performed by **Ecoind et al., 2014** for the treatment of industrial wastewater by using the Chlorella sp. and the removal of COD was obtained 91%. Another study performed by **Lei et al., 2016** in which the removal efficiency of 95.96% of phosphorus was achieved by using Chlorella sp. and Scenedesmus sp.

Gurdev and Pamela, 2012 performed nutrients removal from synthetic municipal wastewater using the co-culture of Chlorella sp., C. vulgaris, Scenedesmus quadricauda and Scenedesmus dimorphus. The result showed the removal of 88% of NO_2 and 49% of NO_3. A consortium of C. vulgaris, S. quadricauda, Euglena gracilis, Ankistrodesmus convolutus and Chlorococcum oviforme was used by **Emienour et al., 2011** for the treatment of municipal wastewater and 99% removal of NH_4^+-N was obtained. **Liang et al., 2009**, checked the effect of Chlorella sp. on nutrients removal from municipal wastewater and resulted in the removal of 85.6% of phosphorus. A study performed by **Jemal and Keneni**, **2018** for the treatment of municipal wastewater by using the mixture of Chlorella sp., Chlamydomos sp. and Scenedesmus sp. as co-culture and the removal of COD was obtained 84%.

4.Research Methodology

This research study was carried out at Civil engineering department of Mehran UET Jamshoro. First of all, the measurement of temperature of water in Civil engineering department at different locations, at different time with specific interval in a peak summer month, July is carried out. We measured the temperature of water in water cooler and washrooms at location 1 and location 2 from 9 am to 3 pm with interval of 2 hours.

Then consumption of water per day calculated by measuring the dimensions of tank (length, width and depth) are and computed routinely the depth of water which is used for drinking, washing and other purposes in Civil engineering department and calculated the volume of water. After that laboratory tests are performed to know the properties of water. The tests which are performed in the laboratory include turbidity of water, total dissolved salts in water, pH value of water, acidity of water, alkalinity of water, chloride concentration and hardness of water. All the data was first recorded in register and then proper fair work and analysis was done.

5.Results

After conducting above mentioned laboratory tests and temperature measures, following results are obtained.

Amount of water consumed in Civil engineering department.

Area of tank (inside) = 23 x 14 = 322

Table 5.1

Day	Difference of depth of water (ft)	Area of tank (sft)	Volume of water (ft^3)	Volume of water in liter.
Monday	0.4	322	128.8	3645
Tuesday	0.3	322	96.6	2733
Wednesday	0.35	322	112.7	3190
Thursday	0.32	322	103	2915

Average 3120 liter of water consumed in 24hours in Civil engineering department.

Characteristics of water

Location No.1 (water cooler A)

Table 5.2

Time	Temperature (°C)	pH	Turbidity (mg/l)	TDS (mg/l)	Chloride (mg/l)	Alkalinity (mg/l)	Acidity (mg/l)	Hardness (mg/l)
9 am	19	8.68	14	360	32	105	2.5	70
11 am	21	8.66	15	360	32	105	2.5	70
1 pm	25	8.7	16	360	32	105	2.5	70
3 pm	25	8.68	14	360	32	105	2.5	70
Average	22	8.68	15	360	32	105	2.5	70

Location No 2 (water cooler B)

Table 5.3

Time	Temperature (°C)	pH	Turbidity (mg/l)	TDS (mg/l)	Chloride (mg/l)	Alkalinity (mg/l)	Acidity (mg/l)	Hardness (mg/l)
9 am	27	8.69	15	360	32	105	2.5	70
11 am	29	8.67	14	360	32	105	2.5	70
1 pm	33	8.65	16	360	32	105	2.5	70
3 pm	34	8.71	16	360	32	105	2.5	70
Average	30	8.68	15	360	32	105	2.5	70

Location No 3 (wash room A)

Table 5.4

Time	Temperature (°C)	pH	Turbidity (mg/l)	TDS (mg/l)	Chloride (mg/l)	Alkalinity (mg/l)	Acidity (mg/l)	Hardness (mg/l)
9 am	30	8.7	19	360	32	105	2.5	70
11 am	33	8.68	18	360	32	105	2.5	70
1 pm	35	8.67	16	360	32	105	2.5	70
3 pm	37	8.71	17	360	32	105	2.5	70
Average	33	8.69	18	360	32	105	2.5	70

Location No 4 (wash room B)

Table 5.5

Time	Temperature (°C)	pH	Turbidity (mg/l)	TDS (mg/l)	Chloride (mg/l)	Alkalinity (mg/l)	Acidity (mg/l)	Hardness (mg/l)
9 am	31	8.65	16	360	32	105	2.5	70
11 am	37	8.66	18	360	32	105	2.5	70
1 pm	40	8.68	17	360	32	105	2.5	70
3 pm	41	8.69	18	360	32	105	2.5	70
Average	37	8.67	17	360	32	105	2.5	70

Table 5.6 (Drinking water standard)

S #	Parameter	Units	Acceptable value	Observed value
01	Temperature	°C	5-10 (Drinking purpose)	20 -30
			15-20 (for washrooms)	30-40
02	Turbidity	NTU	2.5-10	15-20
03	pH		6.5-8.5	8.68
04	Total dissolved solids	mg/l	500	360
05	Chloride	mg/l	200	32
06	Total hardness	mg/l	200	70

6.Conclusion

Based on experimental works following conclusion have been made.

1. The temperature of water remained higher than normally accepted value due to the exposure of water conveying pipe to direct sunlight so students consume more water than the normal during summer because they try to evacuate hot water from the conveying pipe.
2. The normally accepted temperature of water is 15-20°C and in critical conditions maximum value is 30°C but here from the tape water in washrooms it crossed 41^{o}C in hot days of June and July.
3. The drinking water temperature observed from the water coolers was up to 30^{o}C whereas the pleasant temperature is 5 to 10^{o}C.
4. Water losses occurred also due to leakage in pipes of supplying water and hose pipes (during sprinkling and watering to the plants).
5. The length and diameter of pipe which is exposed to sunlight are 120ft and 1.4inch and the water remain in this section is 38litres so it is supposed that an amount of 76litres (approximate) water to be evacuated to get normal water.

7.Recommendations

Based on the research study conclusions following recommendations are made.

1. Water pipe which is exposed to sunlight should be covered with some insulating material like Gunny bags, packing material or foaming sheet.
2. It is calculated that the area required by insulating material would be 45sft. With this area the pipe will be covered but it will not keep the pipe cool so 3 folds (135sft) of material should be provided to keep the pipe cool.
3. Water coolers do not give pleasant cold water to its maximum consumers so water cooler should be recovered with aforesaid material or small fiber tank may be installed under a shade near the water coolers.
4. pH of water was slightly high that is 8.68 so pH value be brought down to the normal level.
5. The average turbidity of source water is 360mg/l was more than (5-10) NTU so turbidity may be reduced at the source; this will improve the cartage life of water filters units of Civil Engineering Department.

8.References

E. Posadas et al., (2014). "Microalgae-based agro-industrial wastewater treatment: a preliminary screening of biodegradability", J Appl Phycol (2014) 26:2335–2345

Ignacio et al., (2009). "Simultaneous nutrients and carbon removal during pretreated swine slurry degradation in a tubular biofilm photo bioreactor". Appl Microbiol Biotechnol (2009) 82:187–194

Ecoind et al., (2014). "Microalgae–Bacteria System for Biological wastewater treatment". Journal of Environmental Protection and Ecology 15, No 1, 268–276 (2014)

Lei et al., (2016). "Microalgae consortia cultivation in dairy wastewater to improve the potential of nutrient removal and biodiesel feedstock production". Environ Sci Pollut Res (2016) 23:8379–8387

Gurdev and Pamela, (2012). "Nutrient removal from membrane bioreactor permeate using microalgae and in a microalgae membrane photo reactor". Journal of Bioresource Technology 117 (2012) 80–85

Emienour et al., (2011). "Use of an algal consortium of five algae in the treatment of landfill leachate using the high-rate algal pond system". J Appl Phycol (2012) 24:953–963

Liang et al., (2009). "Cultivation of Green Algae Chlorella sp. in Different Wastewaters from Municipal Wastewater Treatment Plant". Appl Biochem Biotechnol (2010) 162:1174–1186

Jemal and Keneni, (**2018).** "Microalgae–bacteria consortium treatment technology for municipal wastewater management". Nanotechnology for Environmental Engineering (2019) 4:4

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Integration of Low Head Turbine with Wastewater for Power Generation: A Case Study

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ABSTRACT

The development of low head, low cost and more environmentally friendly Hydropower alternatives is the next step in the evolution of our current Hydropower generation systems. The main objective of this study was the development of a renewable energy system that had a low initial and operational cost along with the development of a functional prototype system and integrating that system with the existing wastewater infrastructure. The methodology was based on the development of both physical and virtually simulated systems, including an Initial Prototype which was optimized by virtual simulation. The findings from the Initial Prototype and test simulations aided in the development of a Final Prototype system which was tested at a flow rate of 1 L/s, giving an output of 0.4 Watts. It was concluded from the on-site observations that the system can be integrated with the existing wastewater infrastructure. The developed industrial scale system operating at a flow rate of 50 L/s gave a power generation potential of 1.57 kW. The project demonstrated itself to be successful in its overarching aim of generating electricity from low-head Hydropower derived from wastewater and integrating the system with the existing wastewater infrastructure without causing much disturbance to the facility.

Keywords:

Final Prototype, GWVPP (Gravitational Water Vortex Power Plant), Industrial Scale System, Test Prototype.

1. Introduction

The advances made by human civilization's over the course of the last century are closely tied in with the exponential rise in global fossil energy demand and consumption. This uptick in energy consumption, propelled by substantial improvements of all key nineteenth-century energy techniques and introduction of new extraction and transportation means has been the fundamental driver behind the industrial and scientific boom of the 21st century, and the technological, social, economic and developmental advances that have followed

since. But this progress has come at the price of the destruction and degradation of the ecosystem. The necessity of mitigating and minimizing the environmental impacts of this rise in energy use, particularly energy with potentially dire outcomes for the planet, is perhaps the greatest trial that faces the planet today.

A global move towards renewable energy is not only inevitable but also is of the utmost importance. International Renewable Energy gency report [1] revealed that the renewable energy resources will be equal to or cheaper than fossil fuels by 2020, with Hydropower proving to be the cheapest at five cents per kilowatt-hour.

Although Hydropower is considered to be a renewable energy resource, its sustainability is sometimes put to question due to the severe environmental impacts that large-scale Hydropower projects entail. Hydropower from wastewater therefore presents itself as a potentially viable value adding energy resource with little to no environmental impact providing a great leap forward in the current renewable energy generation capabilities.

The main objective of this study was the development of a renewable energy system that has a low initial and operational cost. In addition to that, the specific objectives of the project included: 1) the exploration of the potential of wastewater in a way that leads to the development of a functional prototype system; 2) developing a system that is operational at low heads and can be integrated with the existing wastewater infrastructure.

2. Significance in The Local Context

The significance of this project stems from the fact that barring the most highly developed countries, the vast majority of wastewater in the world is released directly to the environment without sufficient treatment as revealed by the World Water Development report [2], with detrimental impacts to the surrounding ecosystem. The city of Karachi alone generates a total of 472 MGD of wastewater, of which the quantity of sewage treated is 50 MGD and the quantity of untreated Sewage which makes it way directly or indirectly to the Arabian Sea without any form of treatment is 417 MGD as reported by [3].

The potential for a system that is capable of harnessing the power of this unwanted wastewater and turning it into a value-adding resource is boundless. That too in an environment where there is a growing need for systems capable of generating emission free renewable energy. Such a system might prove to be a valuable addition to the current renewable energy systems.

3. Literature Review

Murtaza and Zia [4] revealed the total amount of wastewater produced in Pakistan as totaling up to 962 billion gallons. World Bank [5] reports that Pakistan’s ten major cities contribute to more than 60% of the urban wastewater, and only 8% out of this is treated while the rest is being drained directly into River Ravi and Kabul, irrigation canals, vegetable farms and the Arabian Sea. The absence of any real incentives for the industry to treat their effluents is the reason why industrial water pollution remains uncontrolled.

A study conducted by Mohanan [6] in Delhi demonstrated the possibility of using a GWVPP as a method of aerating water combined with power generation for free-flowing drainage water. The power output of a single turbine per day was found to be nearly 820 kWh considering a head of 1.5m and a $3m^3/s$ discharge at an efficiency of 70%. Already existing turbine units reported generation of electricity at $1/watt which is a relatively cheap price when compared to other large power projects. In addition to that the capacity factor of energy generated from wastewater exceeds 95% which is much greater than other forms of renewable energy since drainage water is available all year long. Dhakal [7] demonstrated in his research that GWVPP plants can be installed for roughly the same cost delivering an energy output of 2,850GWh annually, without the damming of an entire river.

A study conducted in Johor, Malaysia [8] detailed the requirements for the optimization of the vortex pool for a GWVPP, which would lead to improved energy conversion resulting in electricity being generated from low water heads. Dhakal [9] analyzed several geometric parameters of the GWVPP system and their effect on vortex formation and energy in a GWVPP. The basin opening was found to be the most critical parameter to be considered during the design of GWVPP. It was recommended that the notch length should be kept long as it would results in the gradual increase in the basin inlet velocity and would therefore prevent unwanted losses.

Thapa and Mishra [10] observed the effect of inlet geometry on the quality of vortex. The study concluded that the triangular inlet geometry path proved to be the most efficient for the vortex flow, producing a symmetric vortex pattern resulting in less imbalanced radial force. Mizanur Rahman [11] studied the impact of the penstock geometry and inlet flow rate on the GWVPP's overall performance. The geometrical configuration of the penstock was found to have a substantial effect on the performance efficiency of the GWVPP, with the results suggesting that the performance of the GWVPP and its output power potential increased significantly with a reduced penstock width due to an increase in the tangential velocity.

Treedet and Suntivarakorn [12] aimed to explore the impact of turbine materials on the overall power generation efficiency of a GWVPP system. The study reported that the turbine material when switched to a lighter weight material with unchanged geometry would contribute towards increased efficiency, output power production and round speed.

4. Methodology

The studies presented in the literature review section followed a similar pattern of virtual simulation testing followed by the development of a physical prototype. Taking cue from those studies, virtual simulation testing was carried out using Autodesk Simulation CFD prior to the physical development of both the Initial Prototype Model and the Final Prototype Model. The virtual simulation results also aided in the result validation of stage of the project. The various components that made up the physical GWVPP prototype model are illustrated in Fig. 1

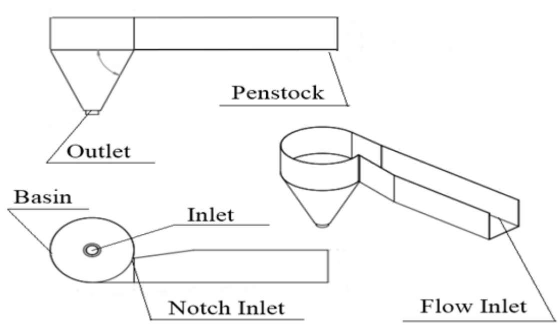

Figure 1: Components of GWVPP Prototype System

The equations governing the performance of the gravitational vortex, are the Continuity Equation and the Navier-Stokes Equations, which are utilized by CFD to simulate the flow of incompressible fluids, the fluid conditions in this study are defined as steady, incompressible, viscous and turbulent flow.

Due to the complexity of the equations and involvement of complex geometry, it was challenging to get an analytical solution directly. Therefore, a 3D model of the Initial Prototype as illustrated in Fig, 2 was modeled using AutoCAD. The said model was virtually simulated for its responses in Autodesk Simulation CFD in order to establish a baseline of results for further development of models in the next phase of the project. This would lead to the identification of the optimum geometry for the proposed system.

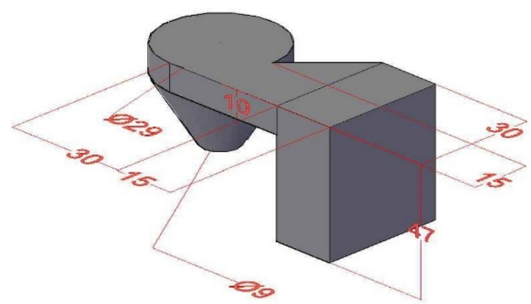

Figure 2: Initial Prototype Model (All dimensions in cm)

The performed simulation was for a vortex velocity distribution associated with a steady flow. The primary assumptions included a steady flow and no slip conditions. The working fluid, water was taken as an incompressible fluid with a fluid density of 998.2 kg/m^3 and a kinematic viscosity of 0.001003kg/m-s. The turbulent model selected to study the flow patterns inside the system was the k-epsilon model. The fluid flow was set to be 1 L/s.

Since it was established by previous studies that the performance of the GWVPP was significantly dependent upon the geometrical parameters of the system therefore the Prototype tested in the initialization phase of the project was optimized in this phase of the project. A total of eight test cases as illustrated in Fig. 3 were developed based on geometrical variations are virtually simulated and the best case was selected for physical development as the Final Prototype.

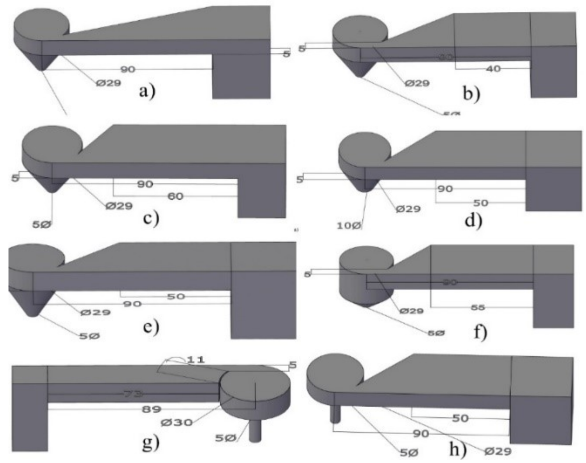

Figure 3: Test Cases with varying Geometrical Parameters (All dimensions in cm)

Selection of the Final Prototype model was based on the following factors. The mean velocities present inside the basin structure, the maximum attained velocity and its location, the velocities present at the two possible location for turbine placement, firstly in the region which lay in between 65-75% of the height of the basin from the top as this was the region identified by the literature to be the optimal for the placement of turbine, secondly the region around the basin inlet and finally the vortex formation since a smooth sustained vortex is more efficient in energy generation.

After each of the virtual testing phases a physical model was developed to imitate the simulation model to validate the virtual simulation results. The physical models once developed was tested inside the Universal Modular Flow Channel as illustrated in Fig. 4 at a flow of 1 L/s.

Figure 44: Universal Modular Flow Channel at NEDUET

The validation procedure for the Initial Prototype consisted of observing the sustainability of the vortex inside the basin and whether the modelling of the said system was suitable inside the given flow channel and the procedure for the Final Prototype consisted of measuring the velocities inside the basin at parametric distances of (5, 10, 15, 20 and 25 centimeters) as illustrated in Fig. 5. and comparing them with the virtually simulated velocities.

Figure 5: Velocity Validation Points

The Final Prototype as illustrated in Fig. 6 was made from a glass fiber sheet. The system was fitted with a runner and shaft assembly along with a DC motor in order to find out the power output of the Final Prototype system.

Figure 6: Final Prototype Model

Results

The virtual simulations of all test scenarios were conducted using Autodesk Simulation CFD. The simulation results provided confidence on the design before investing in the development of the physical prototype model.

The experimental and simulation testing revealed the following issues in the Initial Prototype which needed to be resolved in the next phase of development and testing. The penstock length was too short which created unnecessary turbulence at the notch inlet. The outlet diameter was too big which resulted in decreased velocities at the outlet. The material was flimsy and was not suitable enough to bear the pressures applied on it. The biggest takeaway from the Initial Prototype simulation and testing as illustrated in Fig. 7, was that the vortex was not sustained, the quality of the vortex formed was also low and non-uniform, not suitable for a system depending on the power of the vortex for power generation.

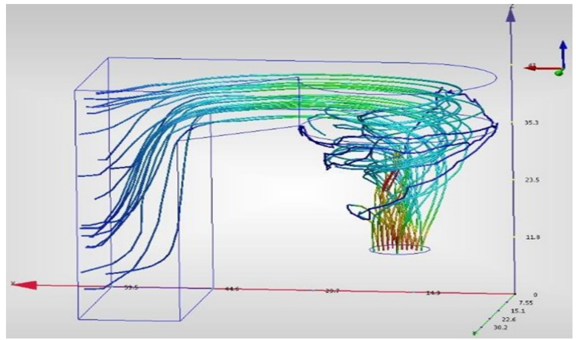

Figure 7: Vortex Formation (Initial Prototype)

The test results reveled that 'Case 8' yielded the best results. The velocities inside the basin within the region of interest where the turbine is to be placed and the average velocity within the system were the highest of all the cases tested. The vortex was also uniform which would avoid unnecessary turbulence within the system which is not suitable for power generation. The summary of the test case results as illustrated in Fig. 8 obtained from the virtual simulation testing of different cases with varying geometrical parameters. The comparison is carried in order to select the optimal model for physical development and experimental testing.

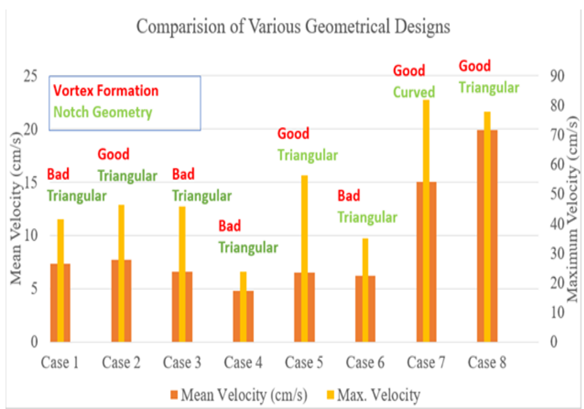

Figure 8: Summary of Test Case Results

The performance of the selected Final Prototype was observed inside the flume conditions at a flow rate of 1 L/s. A runner was developed for the determination of rpm (rotations per minute). The rpms were determined by means of a tachometer. It was found that the runner produced a 100 rpms. A voltmeter was connected to DC motor to find out the potential energy being generated by the system. The energy output was determined to be approximately 0.4 Watts.

The output results obtained from the CFD virtual simulation testing for the Final Prototype required validation by comparing those results with the results obtained through experimental testing of the Final Prototype. The results showed good agreement between the experimental and virtually simulated values with an error of approximately 13 % as illustrated in Fig. 9.

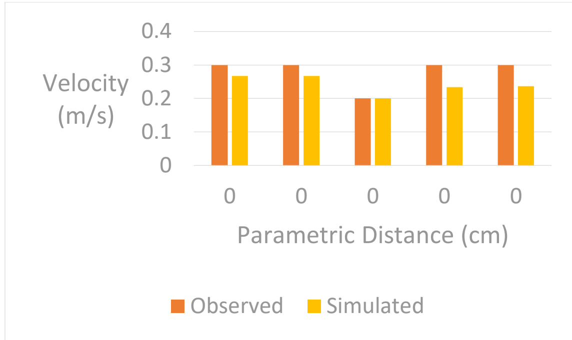

Figure 9: Validation of Outputs

The Effluent Treatment Plant (ETP) at Al-Karam textile mills was selected as a template for the design of Industrial Scale System. The plant has three tanks, and a cooling tower. All the wastewater from the industry is collected in a pit below the treatment plant. Water is then taken to cooling tower 11 m above ground through a pump. This water is taken to Equalization Tank (EQT) 5.5m below the cooling tower as illustrated in Fig. 10 and then to an aeration tank after that the wastewater travels to a sedimentation tank through gravity flows in closed conduits.

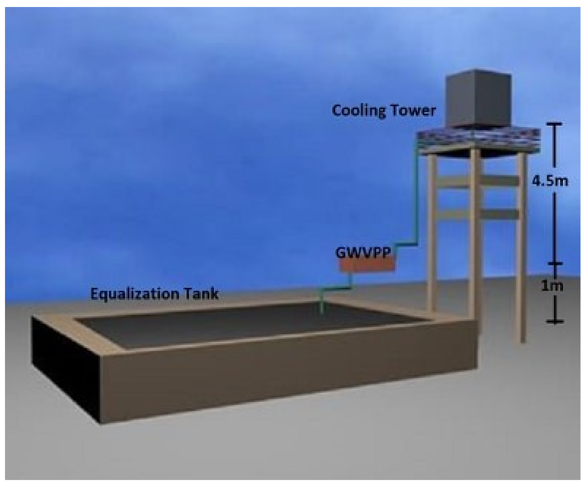

Figure 10: Schematic of the Optimal Location for Industrial Scale System

After much deliberation the optimal location with the most potential for the installation of GWVPP system was after the cooling towers, where the highest potential head was found. A flow of 50 L/s is carried by each 12" diameter pipe. These pipes can act as the inlet flows and deliver water to GWVPP which would go out through the outlet into Equalization Tank through open channels. The high head at this the point will increase the overall velocity of the wastewater which in turn would increase the power generation of the system. Similarly, multiple GWVPP systems can be installed in series as the wastewater makes it way from the cooling tower to the Equalization Tank, each the outlet of each GWVPP feeding the next GWVPP.

The potential up-scale power output was calculated based on the equation Eq. (1) as reported by [13] also utilized by Zotlöterer [14], for their GWVPP system installed and operational in Obergrafendorf, Austria.

$$\mathbf{P = \rho g Q H_{net} \eta} \qquad (1)$$

Where,

P is the power, measured in Watts (W), ρ is the density in (kg/m3), g is the gravitational acceleration constant, which is 9.81m/s2, Q is the flow rate of water in (m3/s), Hnet is the net head. This is the gross head on-site, minus any head losses (losses assumed as 10%), η is the product of all the component efficiencies, which are normally the turbine, drive system and generator.

Therefore, taking the turbine efficiency as reported by Dhakal et al. (2017) as 80%, the drive efficiency as 95% and the generator efficiency as 93% which is typical for small hydro systems. The overall system efficiency would be the product of all the preceding efficiencies resulting in an overall efficiency of 71%.

The power output for the industrial scale system detailed in Section 4.8 is calculated as described in Eq. (2):

$$1000 \times 0.05 \times 9.81 \times 4.5 \times 0.71 = \underline{\mathbf{1.57\ kW}} \qquad (2)$$

5. Conclusions and recommendation

The study was successful in its aim of developing a system that is capable of generating electricity under low heads from wastewater. The energy generated was 0.4 Watts at an average velocity of 19.90 cm/s. Although wastewater itself was not used in the testing of the system, it was observed that a suction was created near the outlet due to sudden change in velocity, which meant that debris would not hinder the performance of the system. The site selection survey for an industrial scale system also proved that the system can be integrated with the existing wastewater infrastructure. The developed industrial scale system operating at a flow rate of 50 L/s gave a power generation potential of 1.57 kW, proving the system to be suitable for off-grid renewable energy generation.

The study also demonstrated that the virtual and physical models showed similar results. The coherence of the physical and virtual models established that CFD can be used for further improved results for future planning.

It is recommended given the findings made by the study, that a GWVPP system can be installed at several places such as rivers, streams or any flowing water body with a low head and high flow rate. It is also suggested that further studies be conducted at different sites which would further establish the potential of a GWVPP system which would prove to be a significant addition to the current sources of renewable energy. This would aid in providing electricity to off grid and remote areas of Pakistan. The turbulence created by the formation of a vortex results in the aeration of water, it is recommended that the extent to which the water is aerated be further explored. A more detailed study could improve upon the results by considering the design of the runner and the electrical circuitry employed for power generation.

6. References

[1] IRENA, "Renewable Power Generation Costs in 2017," International Renewable Energy Agency, Abu Dhabi, 2018.

[2] WWDR, "The United Nations World Water Development Report," UNESCO, Paris, 2017.

[3] KWSB, 2019. [Online]. Available: http://www.kwsb.gos.pk/.

[4] G. Murtaza and M. H. Zia, "Wastewater Production, Treatment and Use in Pakistan," UN Water AIS, Faislabad, 2012.

[5] World Bank, "Pakistan - Country water resources assistance strategy : Water Economy Running Dry," Washington DC, 2005.

[6] A. M. Mohanan, "Power Generation with Simultaneous Aeration using a Gravity Vortex Turbine," *International Journal of Scientific & Engineering Research, Volume 7, Issue 2,* pp. 19-24, 2016.

[7] S. Dhakal, A. B. Timilsina, R. Dhakal, D. Fuyal, T. R. Bajracharya, H. P. Pandit, N. Amatya and A. M. Nakarmi, "Comparison of cylindrical and conical basins with optimum position," *Renewable and Sustainable Energy Reviews,* pp. 662-669, 2015.

[8] H. M. Shabara, O. B. Yaakob, ,. Y. M. Ahmed, A. H. Elbatran and M. S. M. Faddir, "CFD Validation for Efficient Gravitational Vortex Pool System," *Jurnal Teknologi,* pp. 97-100, 2015.

[9] S. Dhakal, A. B. Timilsina, R. Dhakal , D. Fuyal, T. R. Bajracharya and H. P. Pandit, "Effect of Dominant Parameters for Conical Basin: Gravitational Water," in *IOE Graduate Conference*, Pulchowk, 2014.

[10] D. Thapa and A. Mishra, "Effect of Inlet Geometry in the Quality of Vortex Formed Using Vortex Flow Channel," *International Journal of Mechanical Engineering and Technology (IJMET), Volume 8, Issue 5,* p. 515–524, 2017.

[11] T. J. H. a. F. M. T. Md. Mizanur Rahman, "Effects of Inlet Flow Rate and Penstock's Geometry on the Performance of Gravitational Water Vortex Power Plant," Energy Research Unit (ERU), Faculty of Engineering, Universiti Malaysia Sabah, Kota Kinabalu, Sabah, 2017.

[12] P. Sritram, W. Treedet and R. Suntivarakorn, "Effect of turbine materials on power generation efficiency," in *4th Global Conference on Materials Science and Engineering*, 2015.

[13] C. Power, A. McNabola and P. Coughlan, "Development of an evaluation method for hydropower energy recovery in wastewater treatment plants: Case studies in Ireland and the UK," *Sustainable Energy Technologies and Assessments,* pp. 166-177, 2014.

[14] ZOTLÖTERER, "Gravitation Water Vortex Power Plants," 2019. [Online]. Available: http://www.zotloeterer.com/welcome/gravitation-water-vortex-power-plants/economy/.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Study of the Flow Characteristics Around Permeable and Impermeable Spur Dike in an Open Channel

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Abstract
This paper presents the study of local flow characteristics around permeable and impermeable spur dike suitable for Pakistani rivers especially their tributaries in hilly areas of Pakistan. The flow structure is significantly changed by the type of spur dike. The impact of spur dike in the form of permeable and impermeable especially on resulting turbulence flow was studied numerically in an open channel. The Reynolds stress turbulence model was utilized for the present work that was developed by three dimensional (3-D) numerical code FLUENT (ANSYS). After the numerical model was validated, the flow turbulence was studied against 50% porosity of spur dike. The mean velocities with Froude number 0.13 at upstream (US) and downstream (DS) of spur dike showed noteworthy fluctuations across different porosities. The magnitude of mean velocity in spur dike field near embankment was visibly lower than in the mainstream. In the case of impermeable spur dike, large turbulence was observed at spur dike head while with the spur dike having 50% porosity, the turbulence reduced significantly (25%). Based on the results, it is recommended that permeable spur dikes should be preferred over impermeable spur dikes because it will help in reducing the silt deposition inside the spur dike field.

Keywords
Open channel flow, Permeable spur dike, velocity distribution, Numerical modeling, Flow characteristics

1. Introduction

Floods are natural disasters that threaten human lives and impact millions of people all over the world. In Pakistan, floods are major factors, especially in hilly areas, that lead to the destruction of embankments and the degradation of the natural environment. Flash floods in secondary and tertiary rivers cause many damages, particularly the erosion of embankments in Pakistan's hilly areas (Hashmi et al. 2012). Many riverbank protection works are performed to protect the embankments from erosion and spur dikes can be more value-capable and environmentally friendly solution. The spur dikes are installed to improve the depth of channel in the mainstream and to reduce the main streamwise velocity near embankment (King 2009). Moreover, spur dikes are installed to provide a high velocity and turbulence zone in front of spur dike. (Duan 2009) investigated the mean flow and turbulence flow at the front and downstream of the single impermeable spur dike. Experimental results showed that the flow separation zone being stronger than the contracted primary zone. In the presence of impermeable spur dike in emerged and submerged conditions, Teraguchi et al. observed the flow structure and turbulent flow. The experimental and numerical results indicated that high scour depth occurred at spur dike head which created a complex local flow field as compared to other regime of spur dike(Teraguchi et al. 2011).

The hydraulic characteristics of ruble mounds permeable spur dikes were investigated by Zuisen et al. which were installed in the series in an open channel(Zuisen et al. 2005). To study the turbulence effect around single impermeable spur dike Choi and Oh implemented the constant eddy viscosity model. The recirculation regions were also observed just downstream of spur dike which caused to reduce the velocity within the spur dike field(Choi and Oh 2014). Tang observed strong recirculation regions just downstream of single impermeable spur dike after applying the Large Eddy Simulation (LES) model in an open channel. The calculated results showed that these circulating regions lead to silt deposition at the backside of spur dike (Xuelin et al. 2006). The Reynolds Averaged Navier-Stokes (RANS) equations were implemented to observe the flow structure around three river training hydraulic structures(TERAGUCHI et al. 2010). (Kumar and Malik 2016) investigated the mean flow behavior and turbulence behavior around different shapes of spur dikes by using ANSYS workbench. As spur dikes influence the flow velocity and turbulence, therefore, it is important to study the permeability of spur dikes in case of high turbulence at spur dike head for hilly areas and high concentration of recirculation in spur dike field in alluvial rivers.

In the present numerical study, a computational domain with two spur dikes having 0% and 50% permeability in a rectangular channel around which flow behavior (velocity and turbulence) were examined. As in the absence of spur dikes failure of embankments were observed during floods in plain areas and flash floods in hilly areas. To save the embankments from erosion spur dikes are installed. Previous researchers used different angels and shapes of permeable and impermeable spur dikes and find out the high turbulence and velocity at impermeable spur dike head which resulted in high saturation of recirculation zones in spur dike field. This phenomenon of high turbulence and velocity resulted in a high depth of scouring at spur dike head as compared to other regime of spur dike and ultimately partly and sometimes fully failure of impermeable spur dike head observed especially in alluvial areas of Pakistan. The failure of impermeable spur dike head was observed after the sight visit which is at upstream of Sanghar Bridge Taunsa as shown in Fig.1.

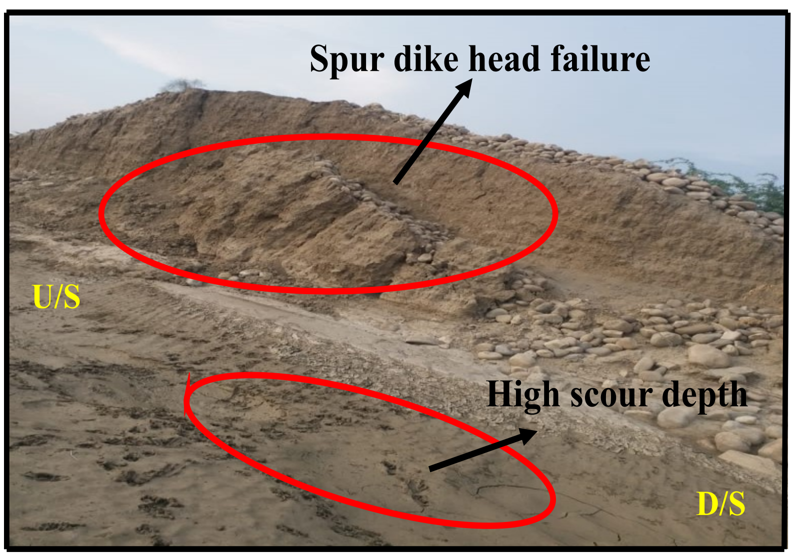


Figure 1: Spur dike head failure due to high scour depth (U/S of Sanghar Bridge Taunsa)

Moreover, with the passage of time volume of silt deposition in spur dike field increased due to low velocity in recirculation zones. The larger volume of silt deposition in spur dike field will reduce the carrying capacity of rivers. Therefore, after providing the 50% permeable spur dike it allows the flow to pass through pores which resulted in the conversion of recirculation zone into parallel streams as well as reduction of velocity and turbulence behavior at spur dike head.

2. Research Materials and Methodology

2.1 Governing Equations

The Reynolds-averaged Navier Stokes equations of continuity and momentum for steady and incompressible flow in an open channel are

The Equation of Continuity

$$\frac{\partial U_i}{\partial x_i} = 0 \tag{1}$$

The Equation of Momentum

$$U_j \frac{\partial}{\partial x_j}(U_i) = \frac{v}{\rho}\frac{\partial}{\partial x_j}\left(\frac{\partial U_i}{\partial x_j} + \frac{\partial U_j}{\partial x_i}\right) - \frac{1}{\rho}\frac{\partial P}{\partial x_i} + (-\rho u_i u_j)$$

(2)

While as u_i ,u_j, and u_k are the velocity components in i, j, and k directions respectively, P is the pressure, $-\rho u_i u_j$ are the Reynolds stresses.

To explain the independent Reynolds stresses transport Eq. (3) includes all variables which explain the partial differential equation used for it as given below (Malalasekera and Versteeg 2007).

$$\frac{\partial R_{ij}}{\partial t} + C_{ij} = P_{ij} + D_{ij} - \varepsilon_{ij} + \prod_{ij} + \Omega_{ij}$$

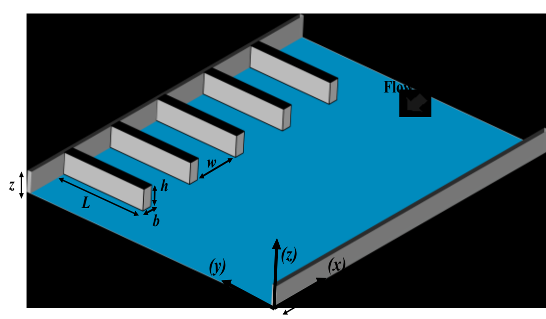
(3)

Where $\frac{\partial R_{ij}}{\partial t}$, C_{ij}, D_{ij}, $\prod_{ij}$, Ω_{ij} are the rate of change of Reynolds stresses, convection term, stresses transport because of diffusion, stresses transport because of turbulent pressure strain, and transport of stresses due to rotation respectively.

2.2 Validation of numerical model

2.2.1 Validation of Streamwise Velocity

The experimental data of (Brevis et al. 2014) was utilized to validate the streamwise velocity (U) at three different positions. All experiments were performed in the fixed flatbed open channel with length= 17 m, width (*B*) = 1.3m, slop (*s*) =0.001 and discharge of 9.75 *L/s* at the Karlsruhe Institute of Technology. The spur dike test segment (24cm×45cm) was provided at the range of 6.5 m upstream from the outlet of the domain. The two emerged impermeable spur dikes were placed in the channel having length (L) 24cm, width (b) 4cm, height (h) 7cm and the distance between two spur dikes (w) were kept 24 cm. The spur dikes placed in the channel were perpendicular to the mainstream. The experimental setup and the location of PIV measurements are shown in Fig.2.



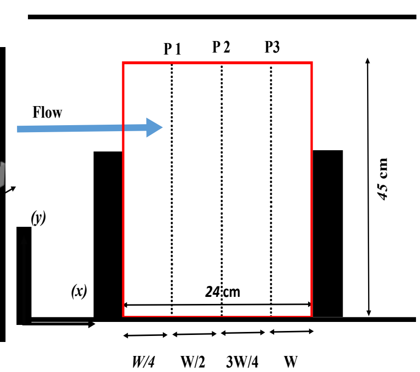
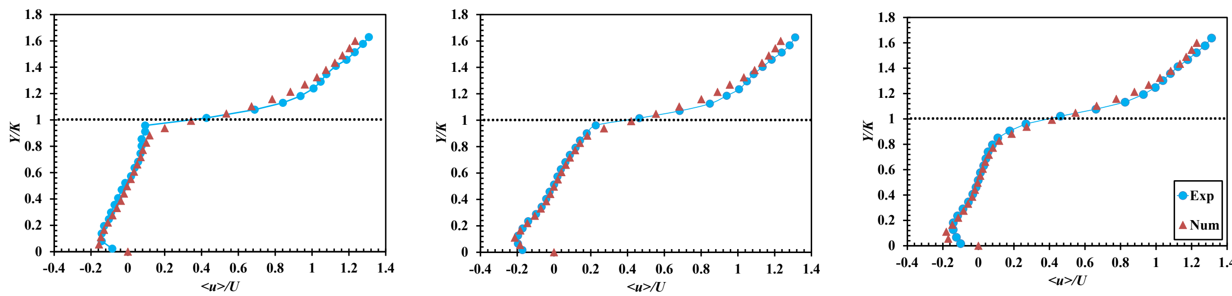


Figure 2: (a) 3-D view of the channel with spur dikes (b) P1, P2, and P3 are the locations of PIV measurements

For the purpose of validation, the geometry needed some simplification. So the selected domain was 56 cm in length and 96 cm in width while keeping all other variables constant. The tri paved unstructured mesh was adopted for the current geometry having about 5.5 million grid points. At inlet and out periodic boundary condition was implemented while as no-slip condition at the bottom wall. The mean streamwise velocity plotted on X-axis was normalized with initial velocity U while as depth flow plotted on Y-axis was normalized with spur dike length (L). The numerical results of mean velocity at different three locations are plotted and relate with experimental streamwise velocities which are given in Fig.3. There was a good agreement between experimental and numerical results, hence showed the validity of the numerical model.



(a) **(b)** **(c)**

Figure 3: Comparison of numerical and experimental streamwise velocity at three positions (a) P1, (b) P2 and (c) P3. The Dashed lines show the head of spur dike.

2.3 Numerical Simulation

The selected computational domain was 56 cm in length, 96 cm in width containing two permeable spur dikes (50% permeability) in an open channel. The rectangular spur dikes have dimensions 24cm×4cm×7cm. The hollow cylinders having a diameter of 2.75 cm are provided in a staggered arrangement within permeable spur dikes. The Reynolds number for flow was kept 10000. Moreover, other hydraulic conditions for computational include discharge $Q = 9.75\ L/s$ while as initial average velocity$U = 0.13\ m/s$. The Reynolds Stress turbulence Model (RSM) computations were utilized for code FLUENT (ANSYS) and to achieve the pressure velocity coupling SIMPLE scheme was applied.

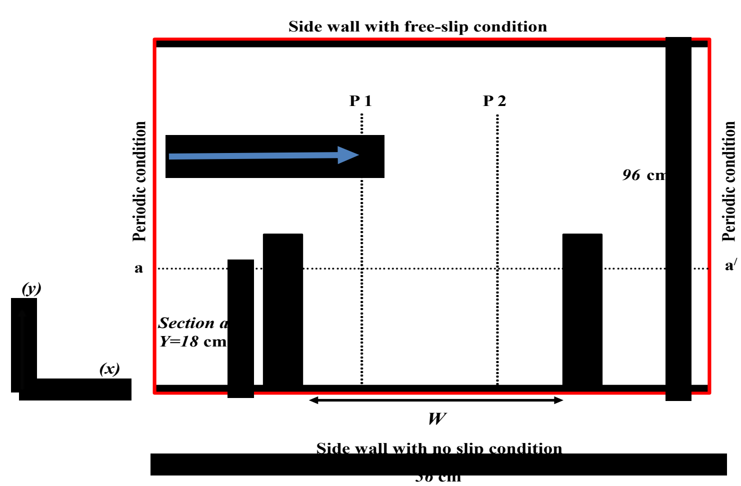


Figure 4: Computational domain with boundary conditions for numerical simulation

There are about two different positions (P1, and P2) which are discussed in detail in the present numerical work which are shown in Fig. 4.

3. Results and Discussion

3.1 Velocity Flow

Two different positions (P1, P2) were considered in the horizontal plane to study velocity behavior for the same hydraulic conditions. In all cases, the spur dikes were kept in emerged condition. At x-axis, streamwise velocity was normalized by dividing it with initial velocity *U* while as at *Y*-axis was normalized by dividing it with spur dike length (*L*). The measuring positions P1 and P2 are located at 22 cm, and 34 cm respectively from the selected computational domain. The measurements of streamwise velocity were taken at half depth of maximum water level (3.5cm) shown in Fig.5.

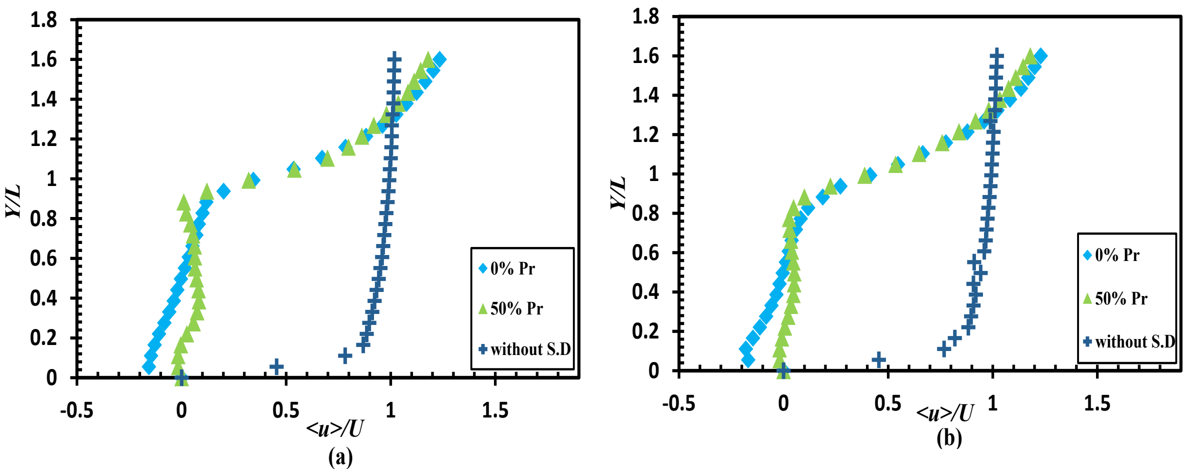


Figure 5: Distribution of streamwise velocity (a) position 1 and (b) position 2 for without spur dikes, 0% permeability (impermeable spur dikes), and 50% permeability (permeable spur dike)

As the flow approaches to spur dikes then mass and momentum exchange behavior occur which resulted in the separation of flow in two regions. In the case of impermeable spur dike, high velocity was observed at the zone of spur dike head at P1 which reduces up to 92% after providing the 50% permeability, while velocity increased at spur dike field up to 223%. In the case of position 2 velocity decreased up to 47% for the zone of spur dike head and increased up to 229% for spur dike field when permeability increased to 50%. Hence, when permeability is provided decrease of streamwise velocity at spur dike head and increase of in spur dike field were observed at both positions P1, and P2.

3.2 Streamlines of mean velocity

The mean streamwise velocity distribution for permeable and impermeable spur dikes are plotted at half of the maximum water depth (3.5 cm) as shown in Fig. 6. The high saturation of recirculation zones was observed because the impermeable spur dike retards the flow velocity in the spur dike field and the high value of velocity in the mainstream. While as in the other regime of spur dike high-velocity fluctuations behavior was observed. The recirculation behavior in spur dike filed may create problems that can be resolved by providing permeability in the spur dikes. The main objective of permeability is to reduce the recirculation behavior of flow within the spur dike field. In the presence of permeable spur dike, the recirculation streams are converted into parallel streams that are responsible to minimize the silt deposition.

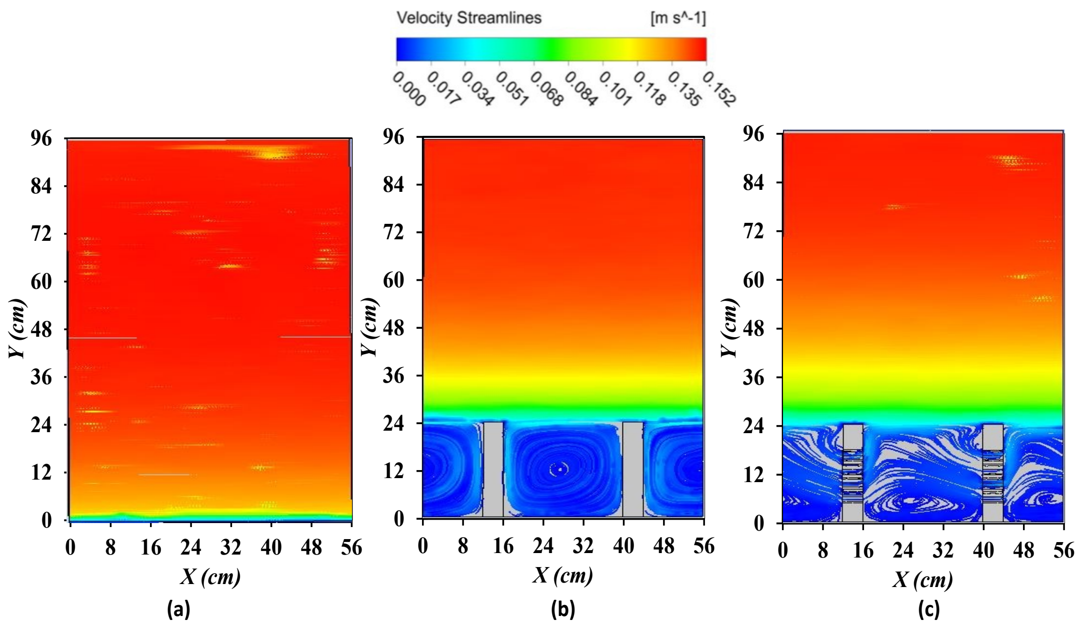


Figure 6: the velocity streams at half of maximum water depth (3.5cm) (a) without spur dikes (b) Impermeable spur dikes (c) 50% permeable spur dikes.

3.3 Turbulent kinetic energy (TKE)

The values of turbulent kinetic energy were calculated in the horizontal plane at half of the maximum water depth. The turbulent kinetic energy (TKE) was calculated at section aa$^{/}$ at y=18cm (Fig.4) to avoid from high value of velocity in the mainstream and very low value of velocity within the spur dike field.

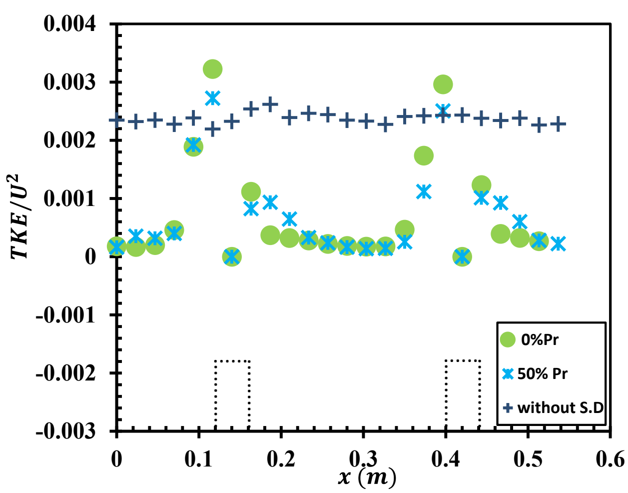


Figure 7: The turbulent kinetic energy for impermeable, permeable and without spur dikes. The dotted lines show the position of spur dikes.

When flow approaches to impermeable spur dikes then high value of turbulence was observed at the face zone of impermeable spur dike which reduces up to 16% just front of upstream spur dike after providing the 50% permeability given in Fig.7. Moreover just back of upstream spur dike low value for impermeable spur dike was observed which are responsible for recirculation zones. So these values can be enhanced by applying the permeable spur dike, which is a controllable factor. The high values of mean velocity and turbulent kinetic energy (TKE) at the face of impermeable spur dikes were observed for both cases, as compared to the back of the spur dike.

3.4 Contours of turbulence intensity

All the contours of turbulent intensity were plotted at the sectional plan aa/ at y=18 cm (Fig.4). The results were concluded against impermeable spur dike and 50% permeable spur dike. There was a significant difference at spur dike head of impermeable spur dike and permeable spur dike having 50% permeability. In Fig. 8 it is cleared that the turbulence decreased as the permeability of spur dike increased because with the increase of pores the flow reduces its velocity. So pores can play a vital role in the distribution of turbulence intensity. Moreover, pores can also provide shelter and food for different types of fish, ultimately can promote ecological system(Hartman and Titus 2010).

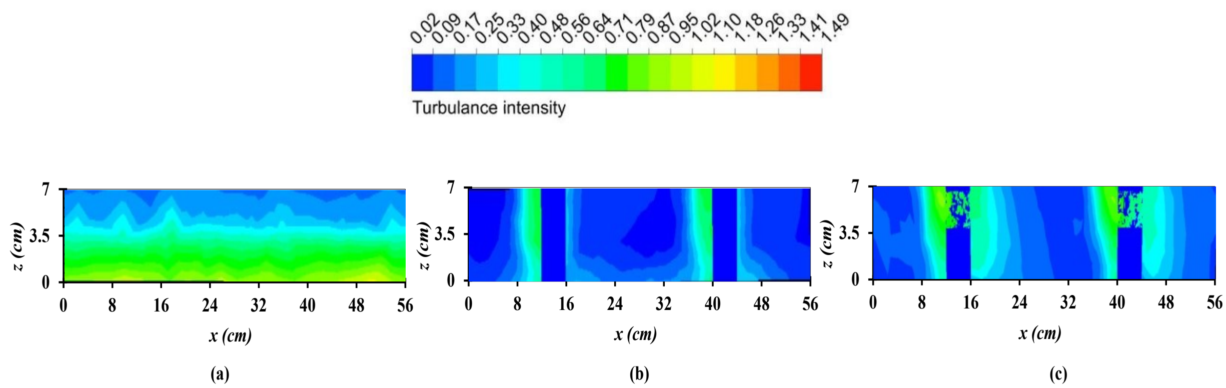


Figure 8: contours of turbulence intensity for (a) without spur dikes (b) impermeable spur dikes (c) 50% permeable spur dikes

4. Conclusion and Recommendation

The Reynolds pressure model (RSM) has been implemented in the current numerical study to explain the flow response by 50% permeable spur dike at half of the maximum water depth (3.5 cm) in a rectangular open channel. The present investigation showed the following conclusions.

- The value of mean velocity (u) reduces 94% at the face zone of permeable spur dike in case of 50% permeability. At high permeability lower value of mean velocity observed at spur dike head.
- As the permeability of spur dike increased it resulted to increase the magnitude of velocity in the spur dike field that ultimately reduces the creation of recirculation zones in the spur dike field significantly.
- The turbulent kinetic energy (TKE) and turbulent intensity showed higher values at spur dike head which may result in the failure of spur dike head. After providing pores in spur dikes reflected to minimize the magnitude at spur dike head.

The conclusion of the present work may become favorable while designing the permeable spur dike for river training works. Therefore, as the natural rivers frequently flow as compound channels, more quantitative analysis is needed in the future to understand the physical flow process containing these permeable spur dike.

Acknowledgments

The authors are grateful to Higher Education Commission (HEC), Pakistan for the provision of CFD software facility at the department of Civil Engineering, University of Engineering and Technology Taxila, Pakistan.

References

Brevis, W., Villalba, M.G., & Niño. Y. (2014). “Experimental and Large Eddy Simulation Study of the Flow Developed by a Sequence of Lateral Obstacles.” *Environmental Fluid Mechanics* 14 (4): 873–93. https://doi.org/10.1007/s10652-013-9328-x.

Choi, S. & Oh, D. (2014). “Finite Element Modeling of Shallow Water Equations For Numerical Simulation ff Flows Near Spur-Dike,” No. July.

Duan, J. G. (2009). “Mean Flow and Turbulence around a Laboratory Spur Dike.” *Journal of Hydraulic Engineering* 135 (10): 803–11. https://doi.org/10.1061/(ASCE)HY.1943-7900.0000077.

Hartman, K. J., & Titus. J. L. (2010). “Fish Use of Artificial Dike Structures in a Navigable River.” *River Research and Applications* 26 (9): 1170–86. https://doi.org/10.1002/rra.1329.

Hashmi. H.N, Siddiqui, Q. T. M., Ghuman, A. R., Kamal, M. A., & Mughul, H. R. (2012). “A Critical Analysis of 2010 Floods in Pakistan.” *African Journal of Agricultural Reseearch* 7 (7): 1054–67. https://doi.org/10.5897/ajarx11.036.

King, H. (2009). “The Use of Groynes for Riverbank Erosion Protection,” no. October. https://doi.org/10.13140/RG.2.1.2773.3203.

Kumar, M., & Malik, A. (2016). “3D Simulation of Flow around Different Types of Groyne Using ANSYS Fluent,” no. 10: 418–26.

Malalasekera, W., & Versteeg, H.K. (2007). “An Introduction to Computational Fluid Dynamics: The Finite Volume Method.”

Teraguchi.H ,Nakagawa.H, Kawaike.K , Yasuyuki Baba.Y and Zhahng.H. (2010) “Morphological Changes Induced by River Training Structures : Bandal-like Structures and Groins.”, no. 53.

Teraguchi, H., Nakagawa, H., Kawaike, K., Baba, Y., & Zhang.H. (2011). “Effects of Hydraulic Structures on River Morphological Processes.” *International Journal of Sediment Research* 26 (3): 283–303. https://doi.org/10.1016/S1001-6279(11)60094-2.

Xuelin, T., Xiang, D., & Zhicong, C. (2006). “Large Eddy Simulations of Three-Dimensional Flows around a Spur Dike.” *Tsinghua Science and Technology* 11 (1): 117–23. https://doi.org/10.1016/S1007-0214(06)70164-X.

Zuisen, L., Michioku, K., Maeno, S. Ushita, T. & Fujii, A. (2005). “Hydraulic Characteristics of a Group of Permeable Groins Constructed in an Open Channel Flow.” *Journal of Applied Mechanics* 8: 773–82. https://doi.org/10.2208/journalam.8.773.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

EFFECT OF SUBMERGED VEGETATION WITH INCREASING PATCH DENSITY ON THE FLOW STRUCTURE

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Abstract
Vegetation consists of various types that usually grow over the floodplains or wetlands in different arrangements. It plays a vital role in influencing the flow speed, sediment transport and other flow characteristics. In this study, the numerical simulation was adopted to examine the flow properties for two different submergence depths i.e. 1< z/h <1.5 and 1< z/h <2.25 in the presence of circular vegetation patches with variable densities along the flow direction. The simulation was performed using FLUENT (ANSYS) integrating Reynolds Stress Model and Navier stokes equation. Three patches with increasing flow blockages were placed on the channel bed to examine the flow structure. The results showed significant reduction up to 127% in longitudinal velocities with submerged and highly submerged vegetation cover. However, stable longitudinal velocities were observed with highly submerged vegetation having larger magnitudes as compared to submerged vegetation. A rise in the turbulence was examined in case of submerged vegetation. A merged wake was observed as the vegetation density increased.

Keywords
Flow structure, variable density, vegetation patches, open channel, turbulence

1. Introduction

Vegetation is the combination of different types of plants including trees, shrubs and grasses, which grow on the wetland or floodplain having influence on velocity and sediment of nutrients transport (Mingliang Zhang, C.W. Li, Yongming Shen, 2013). Vegetation has important contribution in floodplain protection during flood season and speedy flows Neary, (V.S. et al 2012). It plays an essential role in development of ecosystem (Ling-Hui YU et al 2013). The presence of vegetation reduces velocity and shear stress, which ultimately result in the settlement of sediments on channel bed (Jordanova, Angelina A. and Cassandra S. James,2003). Numbers of investigations have been done in the past to observe the impact of

different vegetation configurations on the flow characteristics. Numerous experimental and numerical studies were performed to apprehend the flow behavior, stream-wise flow velocity and turbulent features in the existence of vegetation patches. Neumeier (2007), in his experimental work found out that the magnitude of turbulence reduces due to presence of dense vegetation patch and results in irregular flow on the sides of patch (Neumeier, U. 2007). (Dan Noat 1996) studied the impact of vegetation patch diameter and density on flow properties. He also observed the hydrodynamic impact of fluid in the existence of vegetation elements in the channel bed, sides and corner (Ben Meftah,Mouldi and De Serio,Francesca 2014). Zeng et al. (2013) used Reynolds average Navier Stokes equation (RANS) to simulate the flow velocity along channel and turbulent features through semi-rigid vegetation elements in water channel model (Zeng, Cheng & Li, C. 2014).
On flood plain and embankments, the existence of single vegetation patch is unusual. Patches exists either in series or in dispersed form with variable number of vegetation elements. The dispersed vegetation presence over the floodplain makes the flow-vegetation interaction studies more complex. In the past, several models and their comparisons were carried out on circular patches for variable densities. Zheng bing 2013, Naveed et al. (2018), and others used single patch of circular arrangement to study flow vegetation impact. However, the detailed examination of submerged circular vegetation with variable patch density is found scarce. This study comprises of a detailed analysis of flow behavior in the presence of three patches arranged in a series with increasing density along the flow direction. The flow vegetation interaction in the presence of more than single patch is getting huge attention in hydraulics engineering (Nagare 1995; Anjum and Tanaka, 2020). Hence, it is found necessary to sufficiently explicate the flow structure in the existence of submerged vegetation patches with variable densities.

2. Methodology and materials:

2.1 Experimental setup

The experimental work performed by Ali et al. (2018) was considered significant to validate by numerical model (Ali, Y., Maqsood, F. and Abdul-Rab 2018). The channel used for the experiment was 12.5 m in length, 0.3 m in width and 0.45 m in depth respectively. Vegetation elements consisted of rigid solid cylinders having diameter (d) of 0.64 cm attached over baseboard of the channel. The height of each vegetation element was considered 8 cm. The rigid cylinders were arranged in staggered and circular style. Each of vegetation patches had a diameter of 5.76 cm. The schematic diagram of the experimental model is shown in Figure-1 (a).

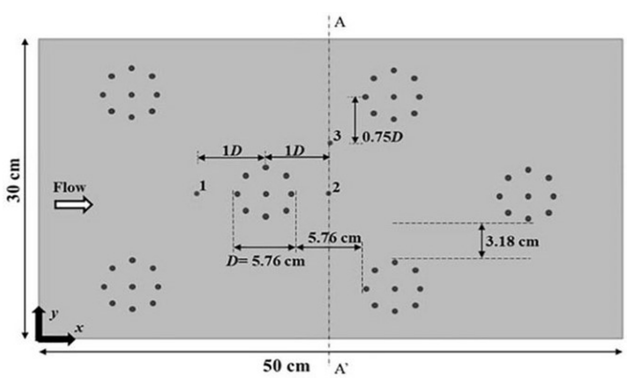


Figure 1(a): Schematic diagram for experimental arrangement of patches (Ali et al 2018)

The experiments were performed for two cases of vegetation including sparse and dense with their flow blockages as 1.2 and 2.3 respectively. The arrangement for the sparse case is shown in Figure-1. Flow rate of 9 L/s was considered for all the experiments. These results obtained from performed experiments were used for validation of numerical model.

Table 1: Hydraulic parameter of experimental setup

Hydraulic parameters, where D and d represent the diameter of the circular patches and vegetation elements respectively. L is the clear spacing between the vegetation patches. h is the height of vegetation element, n represent the number of elements per area of patch, a is the frontal area per volume called vegetation density, aD is the non-dimensional flow blockage, z is the depth of flow, Q is the discharge, U is the mean velocity (U= Q/A), Fr is the Froude number, Re is the Reynolds number for flow (Re = Uz/v)n and Re* is the Reynolds number for cylinder (Re* = Ud/v)

Case	Patch Dia D (cm)	Vegetation element dia d (cm)	L/D	Height of Vegetation element h (cm)	Stem Density n (cm^{-2})	Frontal Area a=nd (m^{-1})	Flow Blockage aD	Flow Depth z (cm)	Flow rate Q (L/s)	U_o (m/s)	Fr	Re	Re*
1	5.76	0.64	1	8	0.3	21	1.2	6	9	0.501	0.653	29,920	3190

2.2 Model setup and boundary condition

The model was simplified and the post-processing and simulations were performed in FLUENT, which is Computational Fluid Dynamics (CFD) tool. Mesh independence of the model was accomplished to validate the quality of numerical model. The domain was meshed using tri-pave unstructured mesh with tetrahedral elements was prepared. Total 1.68 million grid points were obtained. Boundary conditions were applied over the whole domain. Sides of channel, edges of solid cylinder and channel bed were assigned with no slip condition. The free surface of channel was designated with symmetry condition. Periodic conditions were implemented on the inlet and outlet face of the domain. The velocity pressure coupling was attained by employing Semi Implicit Method for Pressure Linked Equations (SIMPLE). A 7-equation Reynolds Stress Model (RSM) was employed for extracting turbulent features of flow. A convergence criterion was set to 1 x 10^{-6} for selected factors. No significant variations in results were observed due to mesh refinement, which showed model independency.

2.3 Validation of numerical model

The data of experiment performed by Ali et al. (2018) was used for the validation of numerical model. The validity of the model was carried out for emergent case (z/h<1) with sparse vegetation arrangement. The stream wise component of velocity (U_x) was normalized with mean velocity (U_o). The comparison of experimental and numerical model was represented [Figure-2] by plotting normalized velocity (U_x/U_o) along x-axis and normalized depth (z/h) along y-axis at critical locations (as shown in Figure-1 (a))

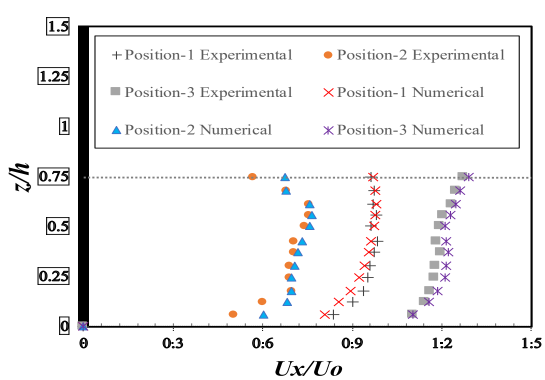
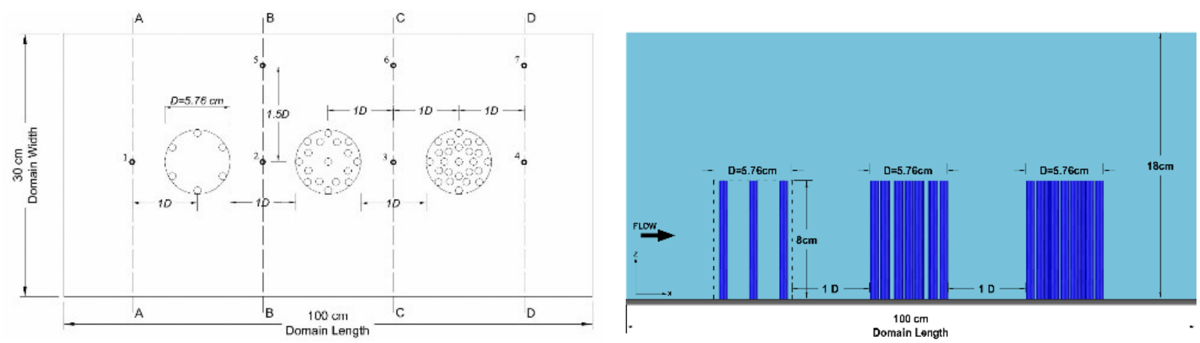


Figure 2: Numerical validation with experimental model

Hence, current model is capable for flow-vegetation interaction study.

3. Conditions for numerical simulation

The purpose of the present study was to investigate the interaction between the varying density patches. The length and width of model was considered 100 cm and 30 cm to achieve the required objectives. The dimeter (D) of each patch was 5.76 cm as used by Ali et al 2018. The diameter (d) of the rigid vegetation element was considered 0.64 cm which lies in the range of 0.1-1 cm that represent the actual stem diameter of emergent vegetation (Y jour Leonard et al. 1995, Wen qi li et al. 2018). The height (h) of each vegetation element was 8 cm. Three circular patches were introduced with in the channel having 6, 17 and 24 number of vegetation elements in upstream, middle and downstream patch respectively. The flow blockages (aD) of the upstream, middle and downstream patch are 0.8, 2.4 and 3.54 respectively. The patches were separated by clear distance equal to diameter (D) of the patch. The vegetation patches were arranged along flow direction with increment in density for submerged and highly submerged conditions. The flow rate of 9 L/s was considered. All the hydraulic parameters have been presented in Table-2 and the model sketches are represented in Figure-3.



(a) **(b)**

Figure 3 (a,b): Schematic sketch of numerical model

4. Governing Equations

The simple form of RANS and continuity equation can be stated as follows:

$$\frac{\partial u_i}{\partial x_i} = 0 \qquad (1)$$

The simplified Navier-Stokes equation used for numerical model:

$$\frac{du_i}{dt} + u_j \frac{du_i}{dx_j} = -\frac{1}{\varrho}\frac{\partial p}{\partial x_i} + g_i + v\nabla^2 u_i \qquad (2)$$

Where t = time, u = fluid velocity, P = fluid pressure, ρ = fluid density, v = kinematic viscosity, ∇^2 = Laplacian operator

The equation used for transport of Reynolds stresses is given as:

$$\frac{\partial R_{ij}}{\partial t} + C_{ij} = P_{ij} + D_{ij} - \varepsilon_{ij} + \prod_{ij} + \Omega_{ij} \qquad (3)$$

Where, R_{ij} = rate change of Reynolds stresses, C_{ij} = transport of convection, P_{ij} = Production rate of Reynolds stresses, D ij = transport of stresses by diffusion, ε_{ij} = Rate of dissipation of stresses, $\prod_{ij}$ = stresses transport, Ω_{ij} = transport of stresses because of rotation.

Table 2: Hydraulic parameter of numerical model

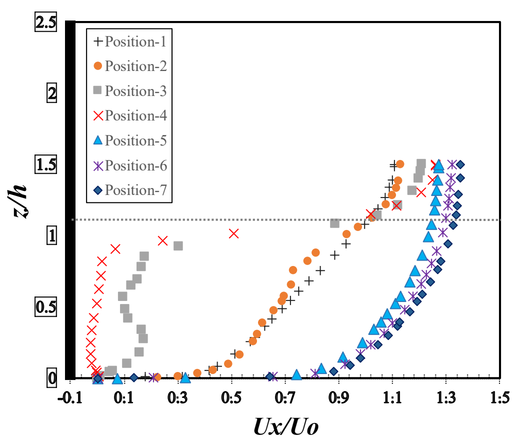
Hydraulic parameters, where D and d represent the diameter of the circular patches and vegetation elements respectively. L is the clear spacing between the vegetation patches. h is the height of vegetation element, n represent the number of elements per area of patch, a is the frontal area per volume called vegetation density, aD is the non-dimensional flow blockage, z is the depth of flow, Q is the discharge, U is the mean velocity (U= Q/A), Fr is the Froude number, Re is the Reynolds number for flow (Re = Uz/v) and Re* is the Reynolds number for cylinder (Re* = Ud/v).

Case	D (cm)	d (cm)	h (cm)	n (cm^{-2})			a=nd (m^{-1})			Flow blockage (aD)			z (cm)	Q (L/s)	U_o (m/s)	Fr	Re	Re*
				u/s patch	middle patch	d/s patch	u/s patch	middle patch	d/s patch	u/s patch	middle patch	d/s patch						
A	5.76	0.64	8	0.23	0.65	0.96	14	42	61	0.8	2.4	3.54	12	9	0.251	0.231	30000	1600
B	5.76	0.64	8	0.23	0.65	0.96	14	42	61	0.8	2.4	3.54	18	9	0.167	0.125	29886	1063

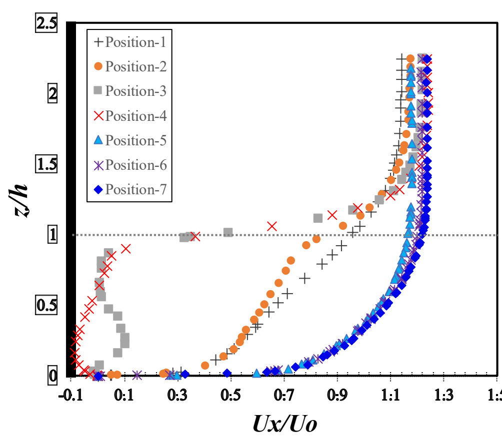
5. Results and Discussion

5.1 Velocity distribution

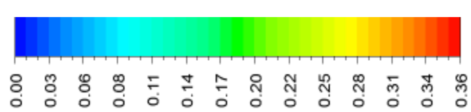
In submerged or highly submerged vegetation case, the flow split into two layers. One layer is obstructed by vegetation while the other layer flows faster over the vegetation height. The velocity increase was observed at downstream of first patch due to the jet generation in between the sparse patch. (Lee, J. H., and V. Chu 2003). However, maximum decrease in velocity was examined at the downstream of last patch due to resistance offered by vegetation patches. The reduction in longitudinal velocities at depth (z/h=0.5) for submerged and highly submerged vegetation case were observed as 100% and 127% respectively at Position-4 which is located at downstream of last patch (aD = 3.54) as compared to initial velocity magnitudes. The longitudinal velocity plots were plotted between normalized velocity (Ux/Uo) and normalized depth (z/h) for submerged vegetation patches of two different flow depths. The plots and velocity contours are shown in Figure-4

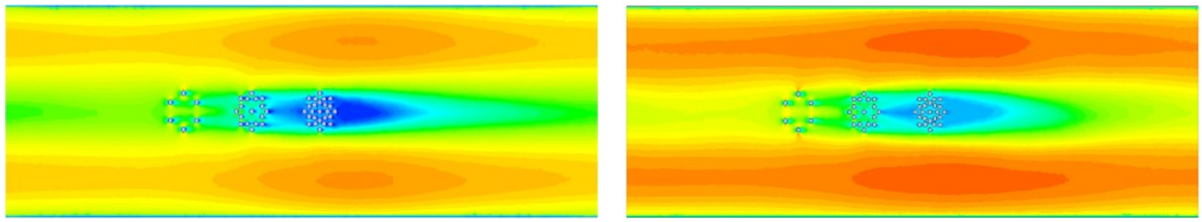


(a) Submerged



(b) Highly submerged





(c) Submerged **(d) Highly submerged**

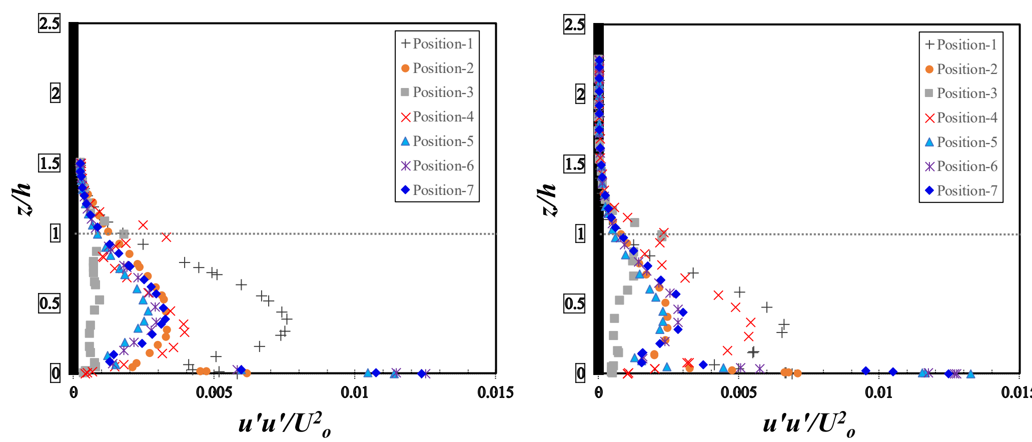
Figure 4 (a,b,c,d): Velocity distribution for submerged and highly submerged vegetation patches

The increase in the vegetation density changes the vertical distribution of velocity at the downstream of the patch. It acquires S shape curve, which was found prominent at Position-3. The velocity has reduced to its maximum at the downstream of last patch (aD=3.54) which eliminated the S shape.

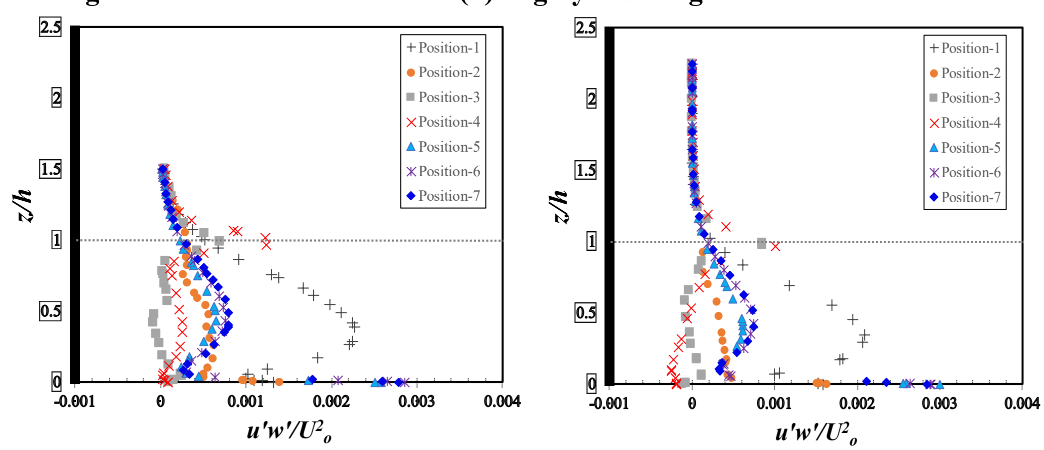
5.2 Turbulent characteristics

5.2.1 Reynold stresses

The maximum value of normal and shear stresses were observed near the bed and at the top of the vegetation (z=h). Sharp fluctuaion has been observed for shear (-u'w') [Figure-5 (c,d)] and normal stresses (u'u') [Figure-(a,b)] at the top of vegetation for both the cases. The existance of Kelvin Helmholtz vortices is due to uncertainity caused by difference in velocities between the two flow layers. The turbulence is directly effected by the genetration of vortices. The normal and shear stresses were found more stable above the vegetation height (z/h=1). The shear stresses at postion-4 were found almost double of the values at Position-3 in case of highly submerged vegetation.



(a) Submerged **(b) Highly submerged**

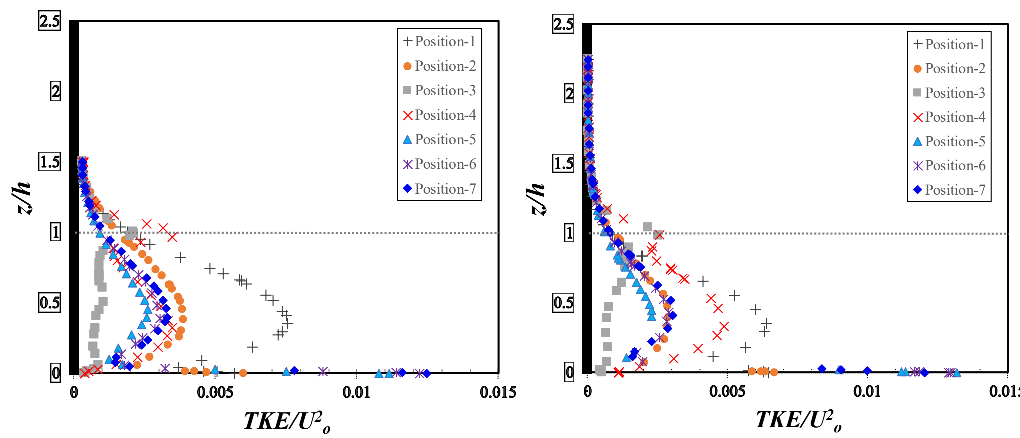


(a) Submerged (b) Highly submerged

Figure 5: Normal and Shear stresses for submerged and highly submerged cases

5.2.2 Turbulent Kinetic Energy

The normalized Turbulent Kinetic Energy (TKE) has been plotted against normalized depth of channel z/h as represnted in the Figure-6 (a,b) for submerged and highly submerged vegetation cases. The turbulent kinetic energy (TKE) was normalized with mean velocity U^2. TKE was found higher in magnitude for submerged case due to higher initial velocity value (U_o=0.251 m/s). Higher energy production was observed near the vegetation patches. The magnitude of turbulent kinetic energy (TKE) was observed higher at Position-1 and Position-2 in submerged vegetation case. However, in case of highly submerged case (z/h=2.25) the values of TKE were observerd higher for Postion -1 and Postion-4. The values then reduced toward the top of vegetation for both the cases in a concav shape. The fluctuations in the TKE values were observed at the top of vegetation. The fluctuatuion at the top of vegetation (z/h=1) was found higher due to increase density of the vegetation patch.

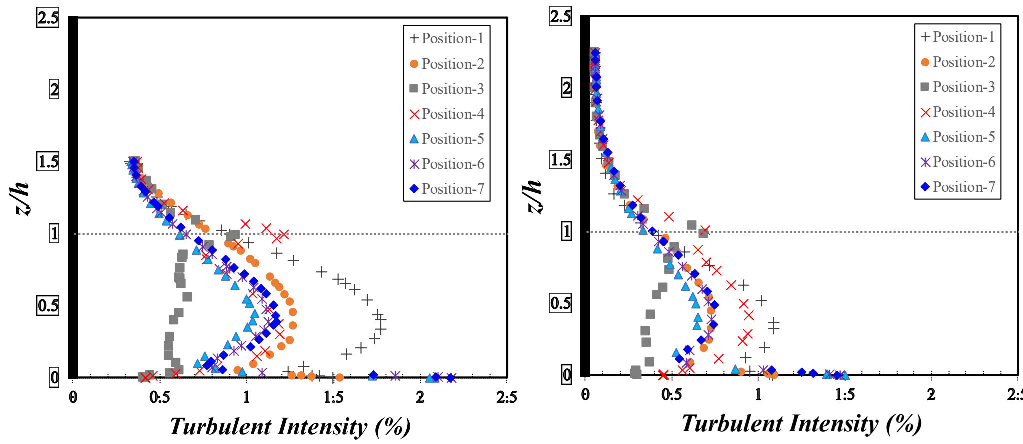


(a) Submerged (b) Highly submerged

Figure 6 (a,b): Turbulent kinetic Energy for Submerged and Highly submerged case

5.2.3 Turbulent Intensity

It is prominent from the graphs [Figure-7 (a,b)] plotted between the turbulent intensity (%) along x-axis and normalized depth of flow (z/h) along y-axis that the percentages of turbulent intensities for submerged case at all the specified locations [Figure-3(a)] were found larger than highly submerged case due to larger initial velocities. The magnitude of Turbulent intenisty (%) at Position-1 was found 67% and at Position-2 73% larger than highly submerged case. Turbulent intensities are directly related with the velocities therefore, the magnitude of turbulent Intensity was found more for higher velocities. (Zhang, Li-Zhi. 2013). The turbulent intensities above the vegetation height (1< z/h< 2.25) in free stream region were found more stable.



(a) Submerged **(b) Highly submerged**

Figure 7 (a,b): Turbulent Intensity for Submerged and Highly submerged case

6. Conclusion

In the current study, the variation of flow properties with variable density was investigated in an open channel using Reynold stress model (RSM). It was evaluated that the vegetation interaction has significant effect on the flow structure, which is connected with the flow velocity and vegetation patch density. It was observed that the velocity has reduced considerably with the increase in vegetation density. Minimum velocities were observed near the bed of channel, which indicate the suitable region for ecosystem and sediment deposition. The magnitudes of velocity for highly submerged case was found 10% less as compared to submerged case. It was observed that the flow properties varied significantly for a specific depth range of 0.6 cm above the vegetation (z/h>1). The induced fluctuations above the vegetation height was observed due to larger momentum exchange between the two layers of flow. Turbulent characteristics were found more in magnitude for submerged case due to higher initial velocity.

7. References

Ali, Y., Maqsood, F. and Abdul-Rab (2018), “Impact of vegetation pattern on flow behavior in an open Channel”. Research Project, 2018, Department of Civil Engineering, University of Engineering and Technology, Taxila, Pakistan

Anjum N, Tanaka N. (2020), “Hydrodynamics of longitudinally discontinuous, vertically double layered and partially covered rigid vegetation patches in open channel flow”. River Res Applic. 36:115 127. https://doi.org/10.1002/rra.3546

Ben Meftah,Mouldi and De Serio,Francesca (2014), “Hydrodynamic behavior in the outer shear layer of Partly obstructed open channels”, Physics of Fluids, volume 26, pages 65-102,

Ghani. U et al, “Investigating the turbulent flow characteristics in an open channel with staggered Vegetation patches” River Res Applic. 2019; 35: 966– 978

Jordanova, Angelina A. and Cassandra S. James (2003), “Experimental Study of Bed Load Transport Through Emergent Vegetation.”

Lee, J. H., and V. Chu (2003), “Turbulent jet and plumes: A lagrangian Approach” Kluwer Academic Publishers, Dordrecht, Netherland

Ling-Hui YU et al (2013), “Numerical investigation of drag force on flow through circular array of cylinders, Journal of Hydrodynamics, Ser. B, Volume 25, Issue 3, 2013, Pages 330-338, online at https://doi.org/10.1016/S1001-6058(11)60371-6.

Li, W., et al. J Hydrodyn (2018). “Effect of vegetation patch density on flow velocity characteristics in an open channel”.
https://doi.org/10.1007/s42241-018-0086-6

Masakazu Tatsuno and Hajime Amamoto, (1995) “Effects of interference among three equidistantly arranged cylinders in a uniform flow" 11Translated from Nagare 14 (1995) 36–50., Fluid Dynamics Research, Volume 22, Issue 5, 1998, Pages 297-315

Mingliang Zhang, C.W. Li, Yongming Shen (2013), "Depth-averaged modeling of free surface flows in Open channels with emerged and submerged vegetation", Applied Mathematical Modelling, Volume 37, Issues 1–2,

Neary, V.S. et al (2012), "Effects of vegetation on turbulence, sediment transport, and stream Morphology". J. Hydraul. Eng. 138(9), 765–776

Neumeier, U. (2007), "Velocity and turbulence variations at the edge of saltmarshes" Continental Shelf Research, 27 (8), 1046-1059

Y JOUR Leonard, Lynn A. et al.(1995) "Flow hydrodynamics in tidal marsh canopies, Journal Limnology and Oceanography", Volume 40 https://doi.org/10.4319/lo.1995.40.8.1474.

Zhang, Li-Zhi. (2013), "Heat and Mass Transfer Across a Hollow Fiber Membrane Bundle" 10.1016/B978-0-12-407782-9.00007-1.

Zeng, Cheng & Li, C. (2014), "Measurements and modeling of open-channel flows with finite semi-rigid Vegetation patches" Environmental Fluid Mechanics. 14. 10.1007/s10652-013-9298-z.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

SFRC SPECIMENS UNDER INCREASING COMPRESSIVE LOADING RATES

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Abstract

Concrete fails abruptly after reaching its maximium compressive and tensile strength due to its inherent brittle property. As it is weak in resisting tensile stresses, steel reinforcing bars are used in the construction of the structural memebers to carry tension and in addtion to provide ductility. However, the use of steel fibers has also become increasingly popular in the prepartion of structural concrete over last several decades. The studies have shown that with the inclusion of steel fibers not only the resisting capacity of concrete increases in compression and tension but the britlle behaviour associated with concrete also decreases. Similar to the plain concrete specimens, published experimental studies have shown that with increase in the loading rates the behaviour of steel fiber reinforced concrete specimens also changes when compared with same under static loading conditions. However, these studies does not define the qualitative explanation which causes such abrupt change in the behaviour. Therefore, the study presented herein focuses on investigating numerically in detailed the behaviour of steel fiber reinforced concrete specimens under increasing compressive loading rates using non-linear finite element analysis software ABAQUS. It was found that the behavior exhibited by these specimens under high rate compressive loading represent the structural response rather than the material behavior.

Keywords
Steel fibers, Dynamic response, Finite element, Concrete, Compressive loading

1. Introduction

Due to the inherent brittle property of concrete, it is typically reinforced with the steel bars in the construction of the structural elements. The design codes [Committee (2005), Eurocode-EC2 (2004)] used for the construction industry does not consider the tension stiffening of the concrete in the design of the structural elements mainly due to; as it only considered (i) linear behavior and as (ii) the capacity of the concrete in tension is very small. Over the last few decades, efforts have been made to increase the tensile strength of concrete by altering its property. For example, with the use of Engineered Cementitious Concrete (ECC), and steel fibers etc. Number of experimental studies have been carried out to study the behavior of steel fibre reinforced concrete under static and dynamic loading. Based on these studies in general, it was found that with the inclusion of steel fibers the peak compressive and peak tensile strain at failure increases. Furthermore, the inclusion of steel fibers also proved to increase the inherent brittle property of the concrete. Based on the high rate loading experimental studies, the behavior of steel fiber reinforced concrete significantly differs from that observed under static loading.

The behaviour of steel fiber reinforced concrete (SFRC) specimens under increasing compressive loading rates of 42/s to 99/s was studied by Wang *et al.* (2008). Based on the study, it was observed that specimens having 3% of steel fibers resulted in higher strength in the compressive strength as compared to the specimens with 6% steel fibers. Similarly, Yoo *et al.* (2015) also investigated the response of normal and high strength SFRC specimens having steel fiber contents of 0%, 1%, 1.5% and 2% under both static and high rate compressive loadings and found that steel fibres affect significantly both stresses and strains at failure increases. The behavior of hooked and straight SFRC specimens under increasing compressive loading rates was also investigated experimentally by Rostasy & Hartwich (1985) . It was observed that 0.75% hooked fibers SFRC specimen exhibited slight improvement in the compressive strength as compare to other specimens investigated.

Although, above studies highlighted significant change in the behavior of SFRC specimen under high rate compressive loading but does not provide the causes which results in this behavior. Therefore, the study presented herein focuses on investigating numerically in detailed the behaviour of steel fiber reinforced concrete specimens under increasing compressive loading rates using non-linear finite element analysis software ABAQUS. It was found that the behavior exhibited by these specimens under high rate compressive loading represent the structural response rather than the material behavior.

2. Numerical Investigation

In order to investigate, the behavior of steel fibre reinforced concrete specimens under static uniaxial compressive loading initially test results of Rostasy & Hartwich (1985) were used to validate non-linear finite element analysis. Rostasy & Hartwich (1985) investigated the behaviour of steel fibre reinforced concrete specimens under uniaxial compression prepared with 0.75% steel fibre. The plain concrete compressive strength of 25MPa was used in this investigation.

2.1 Validation of FE analysis

To investigate the behaviour of steel fibre reinforced concrete specimen under uniaxial compressive loading, a cylindrical specimen having a diameter of 100mm and height of 200mm was used. The specimen was fixed from its base and the load was applied from the top of the specimen with the help of steel cap using displacement control load. The steel cap used has a diameter of 100mm and height of 25mm.

Figure 1 shows the comparison of the stress-strain curves exhibited experimentally and predicted numerically for the case of steel fibre reinforced concrete specimens. As can be seen that excellent agreement has been found between the experimental results and the FE predictions. More specifically, similar predictions for the peak stress was observed, however, difference in the post peak response was observed between experimental results and the FE predictions. This is due to the fact that the concrete failure was not explicitly modelled in FE analysis, as main aim of the study was to focus on the specimen peak compressive stress. Figure 2 shows the deformation profile of specimens at failure. It was observed that whole specimen reacts to the applied load as typically observed during uni-axial compression tests.

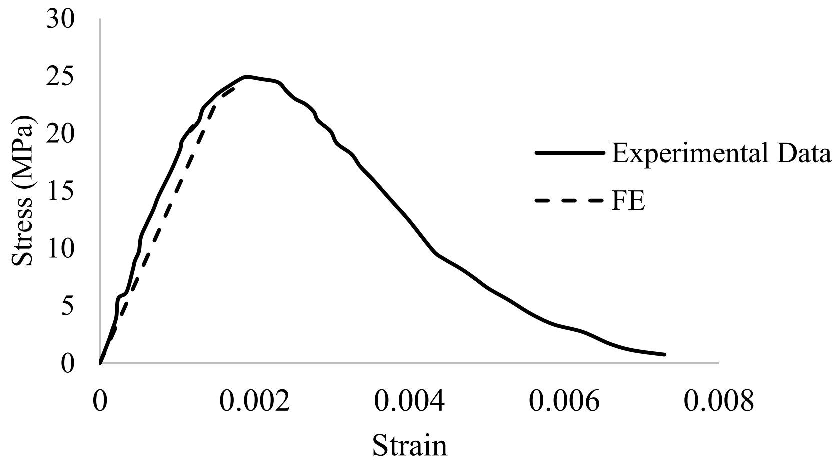


Figure 1: Comparison of the stress-strain curves exhibited experimentally and predicted numerically by steel fibre reinforced concrete specimens.

2.2 Behaviour of SFRC specimens under increasing compressive loading rates

In order to study the behaviour of SFRC specimens under increasing loading rates a finite element investigation was carried out using same FE model having same material properties as described in section 2.1. The behaviour of SFRC specimens were investigated under increasing compressive loading rates of 10/s, 100/s, 250/s, 500/s and 1000/s. For this purpose, non-linear finite element implicit analysis was carried out using ABAQUS.

Figure 3 shows the dynamic increase factor (DIF) in terms of SFRC compressive strength exhibited at higher loading rates to that under static loading rate. In general, it was observed that with the increase in the loading rate the DIF also increases. This observation is similar to the experimental studies published previously as described in section 1. Based on above, it can be concluded that the behaviour of the SFRC specimens under increasing loading rates differs significantly as compared to its counterpart for the case of static loading.

Figure 4 shows the variation in strain rate measured at top, bottom and center of the cross-section with respect to time. It was observed that the strain rate investigated numerically was significantly higher than the applied strain rates. Furthermore, it was also observed that as the applied loading rate increases, the behavior become more localized, thus representing structural behaviour rather than the material response. Figure 5 shows the deformed profile under increasing strain rates at failure for SFRC specimens. In general, it was observed that with the increase in the strain rate, the behaviour became more localized as the maximum deformation occurred at the top of the cross-section irrespective of concrete strength used as shown in Figure 5. Therefore, it can be concluded again that response of these specimens represents the structural behaviour rather than the material behaviour.

Figure 6 shows the damage profile of SFRC specimens under increasing loading rates. As expected, it was observed that with the increase in loading rates, the damage caused to the SFRC specimens decreases significantly and became localized near the steel caps.

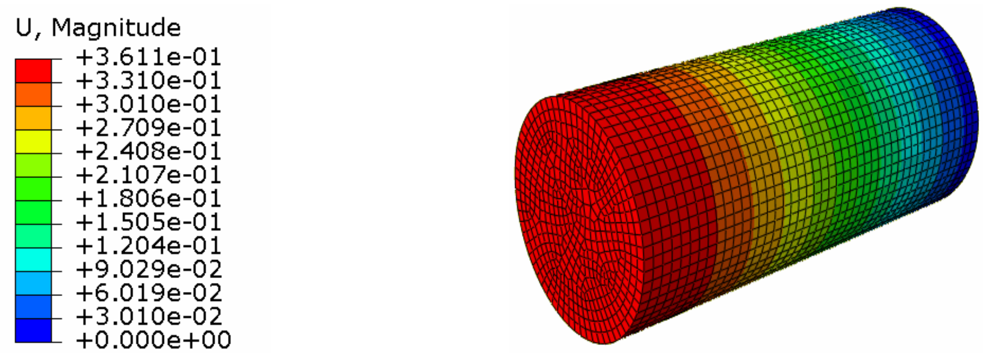


Figure 2: Deformed profile of SFRC specimen subjected to uniaxial static compressive loading.

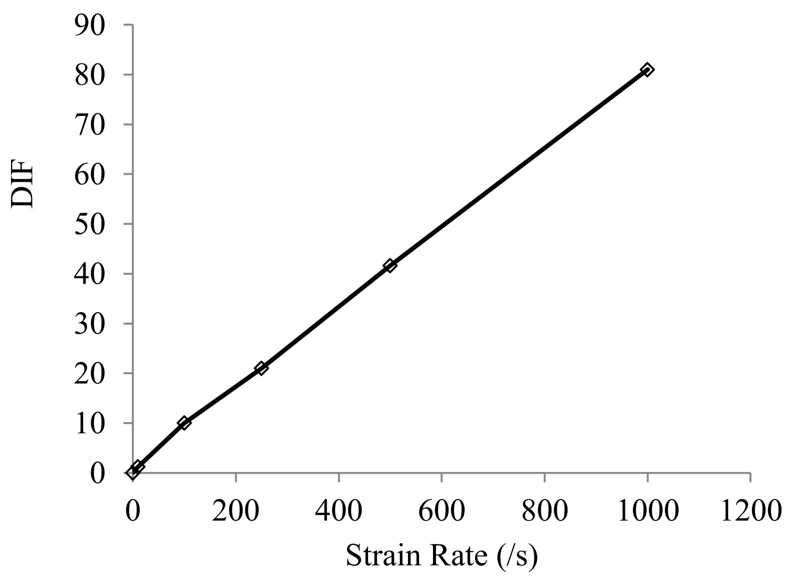
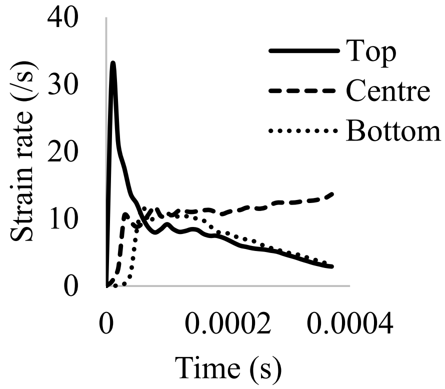
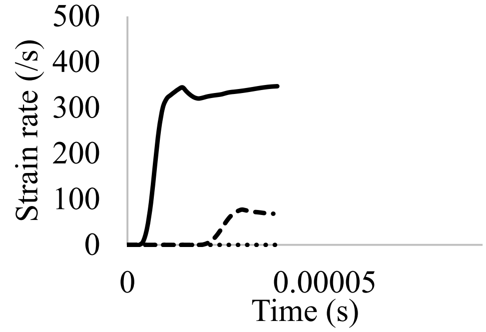


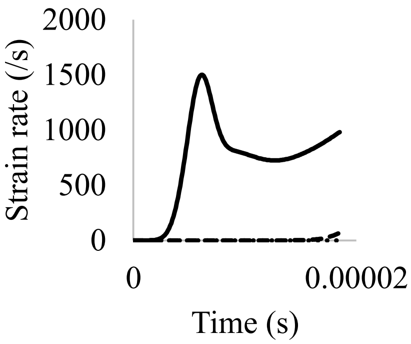
Figure 3: Variation of DIF representing compressive strength of SFRC specimens under high loading rates to that under static loading with respect to increasing strain rates.



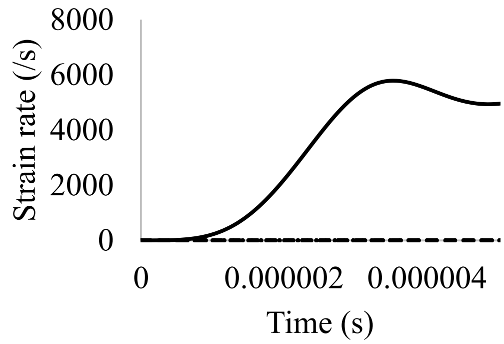
(a) 10/s



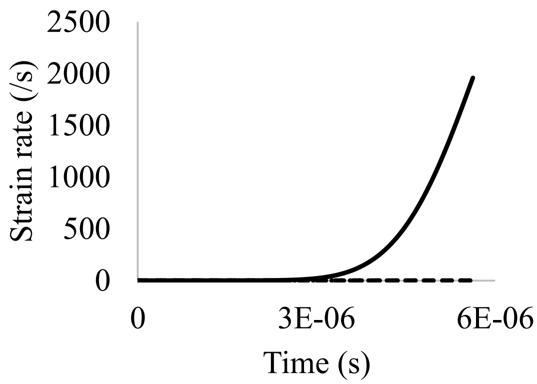
(b) 100/s



(c) 250/s



(d) 500/s



(e) 1000/s

Figure 4: Strain rate with respect to time measured at top, center and bottom fibre of SFRC specimen having compressive strength of 25MPa.

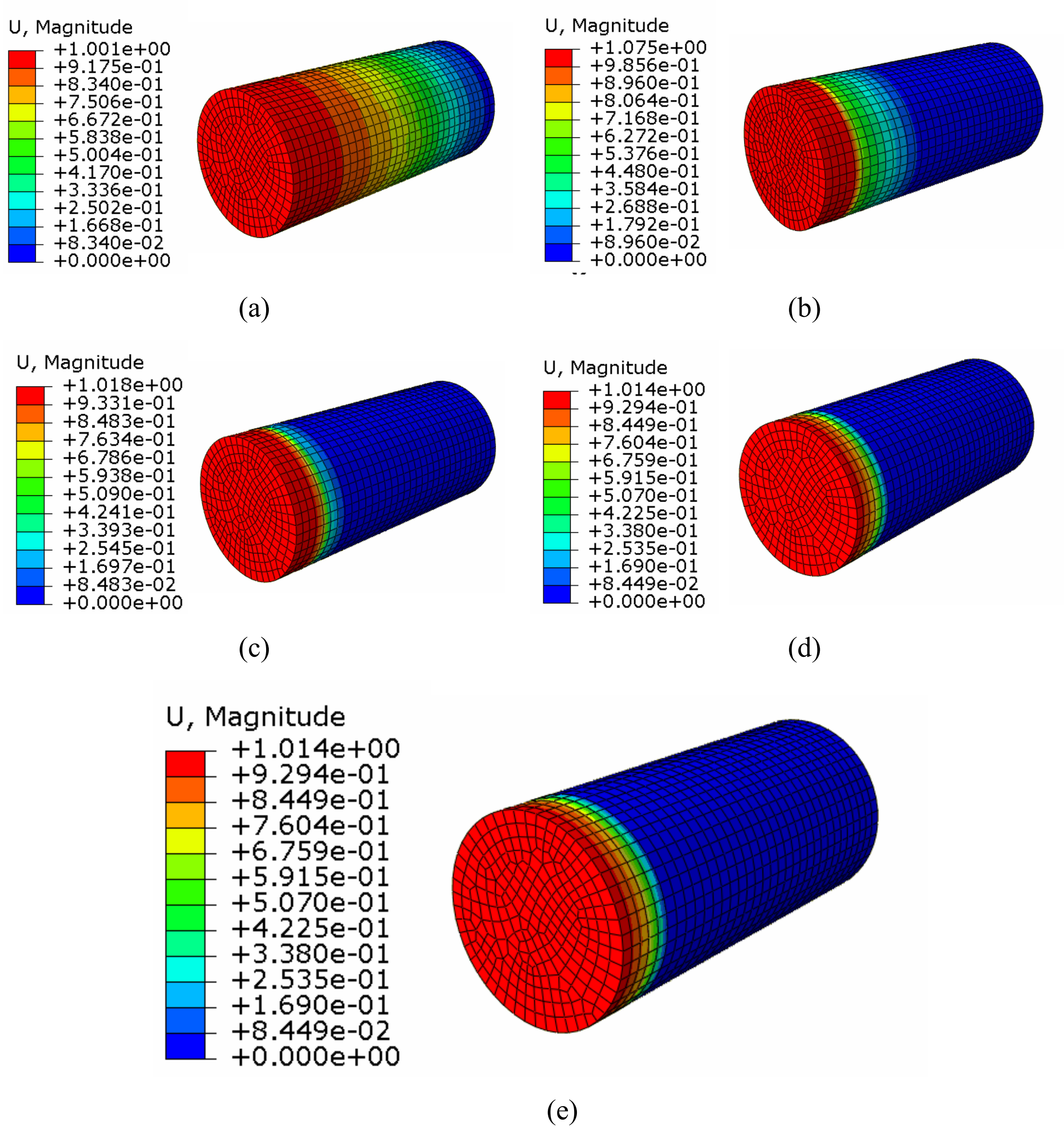
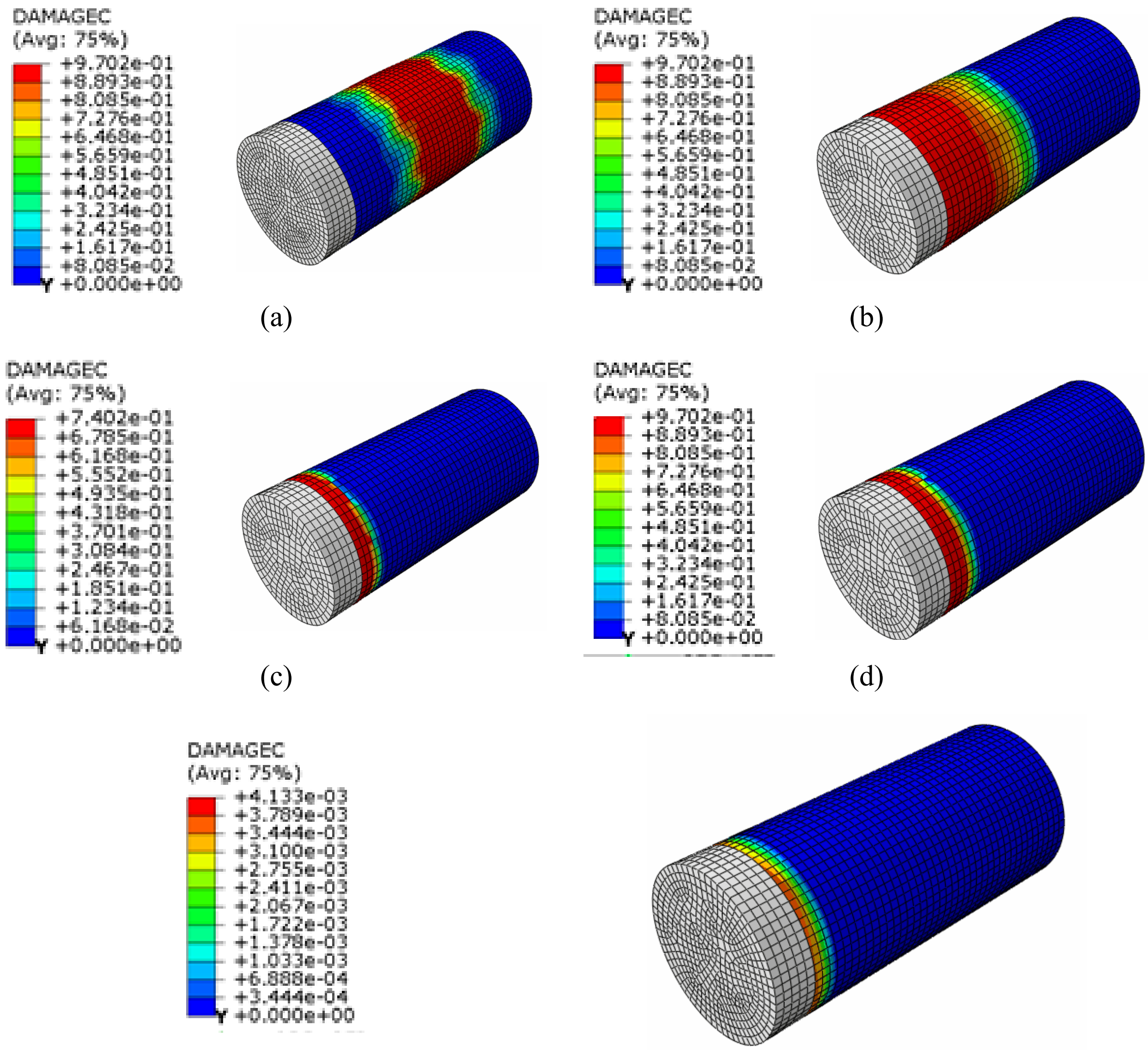


Figure 5: Deformed profile for SFRC specimen having compressive strength of 25MPa under increasing sttrain rates of (a) 10/s, (b) 100/s, (c) 250/s, (d) 500/s and (e) 1000/s.



(d)

Figure 6: Damage in terms of compression crushing caused to SFRC specimen having compressive strength of 25MPa under increasing strain rates of (a) 10/s, (b) 100/s, (c) 250/s, (d) 500/s and (e) 1000/s.

3. Conclusions

Based on the detailed FE investigation studying the behaviour of steel fibre reinforced concrete specimens under static and high rate compressive loading following conclusions were drawn:

(i) The behaviour of the SFRC specimens under higher rate compressive loading was found to be significantly different as compared to that under static loading.
(ii) The increase in the loading rate also results in significant increase in the dynamic increase factor (DIF).
(iii) With the increase in the loading rate the behaviour exhibited becomes more localized.

REFERENCES

Committee, A. Building code requirements for structural concrete (ACI 318-05) and commentary (ACI 318R-05). 2005. American Concrete Institute.

Eurocode-EC2 2004. *Eurocode 2: Design of Concrete Structures: Part 1-1: General Rules and Rules for Buildings*, British Standards Institution.

Rostasy, F. & Hartwich, K. (1985). Compressive strength and deformation of steel fibre reinforced concrete under high rate of strain. *International Journal of Cement Composites and Lightweight Concrete,* 7**,** 21-28.

Wang, Z.-L., Liu, Y.-S. & Shen, R. (2008). Stress–strain relationship of steel fiber-reinforced concrete under dynamic compression. *Construction and Building Materials,* 22**,** 811-819.

Yoo, D.-Y., Yoon, Y.-S. & Banthia, N. (2015). Flexural response of steel-fiber-reinforced concrete beams: Effects of strength, fiber content, and strain-rate. *Cement and Concrete Composites,* 64**,** 84-92.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Influence of Asymmetry on Local Damage Response of Plan-Asymmetric Reinforced Concrete Structure

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Abstract
This research focuses on the abnormal behaviour of the local seismic response of corner column at the flexible edge (CCFE) of a plan-asymmetric reinforced concrete (RC) structure. A quarter scaled plan-asymmetric structure was experimentally tested under progressive seismic excitations and internal seismic damage response at the base of CCFE was monitored using Fibre bragg grating (FBG) strain sensors. Experimental and numerical investigations concerning to the behaviour of the local damage response of CCFE are presented to highlight the influence of asymmetry on varying local seismic demands within the same structural component. To validate the experimental findings, a calibrated Finite Element (FE) model was established in ABAQUS. This research concludes that CCFE in plan-asymmetric structures require substantial design redundancy and special seismic detailing to resist intense seismic shaking.

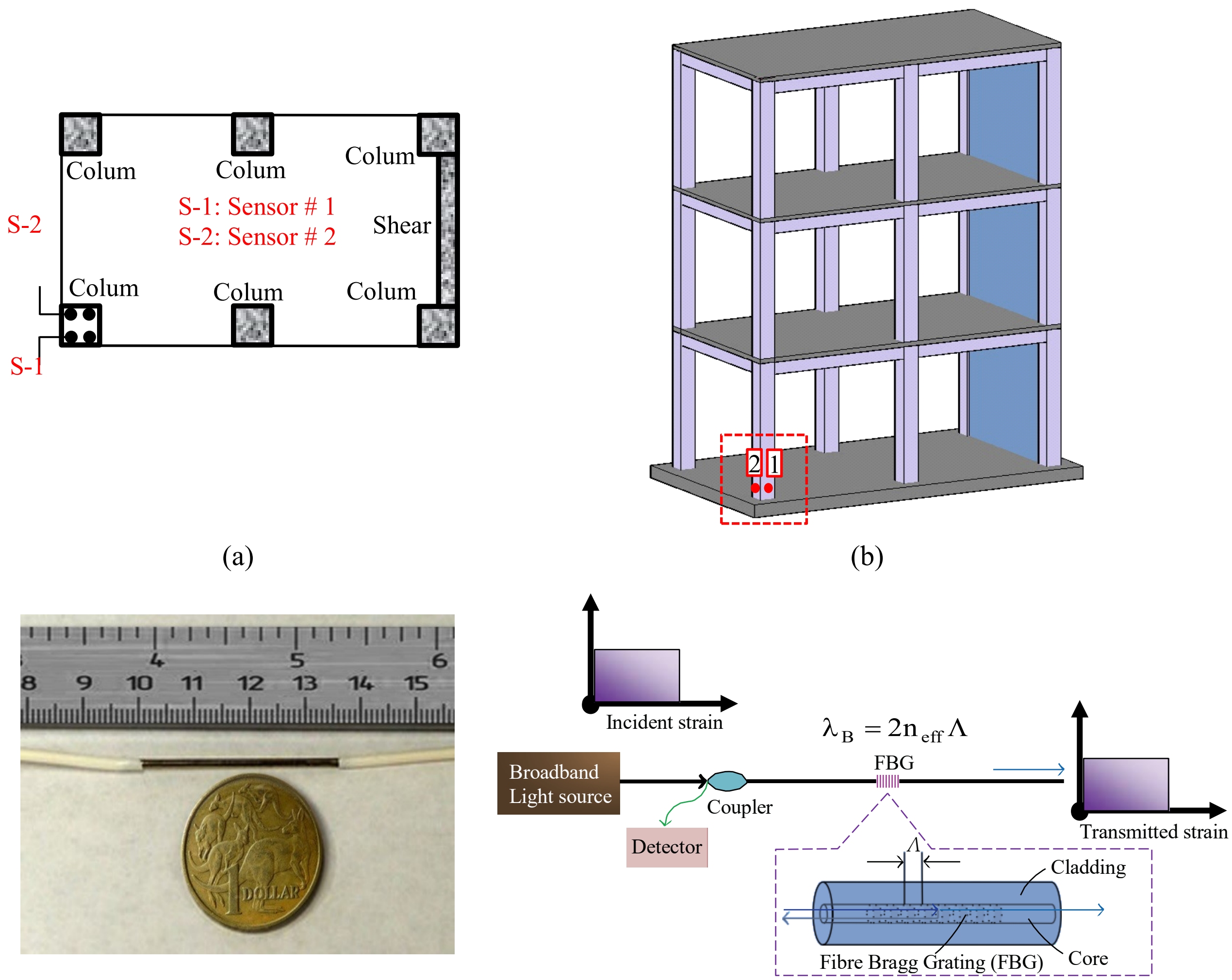
Keywords
Shake table test; Progressive seismic loading; Local seismic response; Residual strain

1. INTRODUCTION

Recent major earthquakes prove the fact that reinforced concrete (RC) structures are prone to collapse under seismic actions (Takewaki et al., 2011). The reasons of structural collapse can be evaluated from several aspects. Some of these aspects can be ground motion uncertainties (Takewaki, 2005, Li et al., 2004b) and asymmetry of the structure (Hejal and Chopra, 1989, Alam et al., 2016, Zhang et al., 2016, Alam and Zhang, 2019a, Alam and Zhang, 2019b, Alam and Zhang, 2019c, Alam et al., 2020). However, majority of the previous research is limited to the global seismic response of the asymmetric structures. To date only a handful of experimental studies have been reported to address the damage behaviour of asymmetric structures (Pearson and Delatte, 2005, Lu et al., 2016, Martinelli and Filippou, 2009, Ghorbanirenani et al., 2011). The reason for this research gap is that damage response investigation needs a careful handling for accurate response measurement. In this research, FBG strain sensors were implemented for internal response measurement as FBG strain sensors have the ability of periodical variation in the index of refraction of the optical fibre core (Kersey et al., 1997, Li et al., 2004a). In the light of the presented background, this research is aimed to investigate seismic damage response in a critical structural component under the influence of structural asymmetry.

2. EXPERIMENTAL MODEL

The model design, geometric configuration and reinforcement description can be found in the companion paper (Zhang et al., 2018). The scope of this research is limited to the damage behaviour at the base of the flexible edge of plan-asymmetric structure, therefore, the structural local response obtained from sensor # 1 and sensor # 2 (Figure 1a and 1b) will be discussed here. For deployment of the FBG sensors, several protection measures were taken in order to obtain accurate measurement of the response. These measures have been explained in the literature in detail (Zhang et al., 2018). The basic principle of FBG sensors and their locations for this study has been demonstrated in Figure 1.



(a) (b)

(c) (d)

Figure 1: Selected column at the flexible edge of the structure for strain monitoring (a). Location of FBG sensors under consideration (b). Location of FBG sensors at other locations for validation of experimental results with numerical results (c). Stainless steel tube-packaged FBG sensor. (d). Schematic diagram of stainless steel tube-packaged FBG sensor.

3. DYNAMIC CHARACTERISTICS OF THE STRUCTURE

Figure 2 illustrates the variation in the dynamic properties of the structure investigated after each loading state. It is evident that the frequency reduction is only higher in the 1st mode as compared to the 2nd mode where the structure experienced relatively lower frequency reduction of 35% after the final loading state. Conversely, looking at the damping ratio of the structure, it can be seen that damping ratio increased gradually with decrease in the frequency of the structure. However, the damping ratio augmentation is relatively equal in 1st and 2nd modes having an approximate augmentation of 80% and 85% respectively.

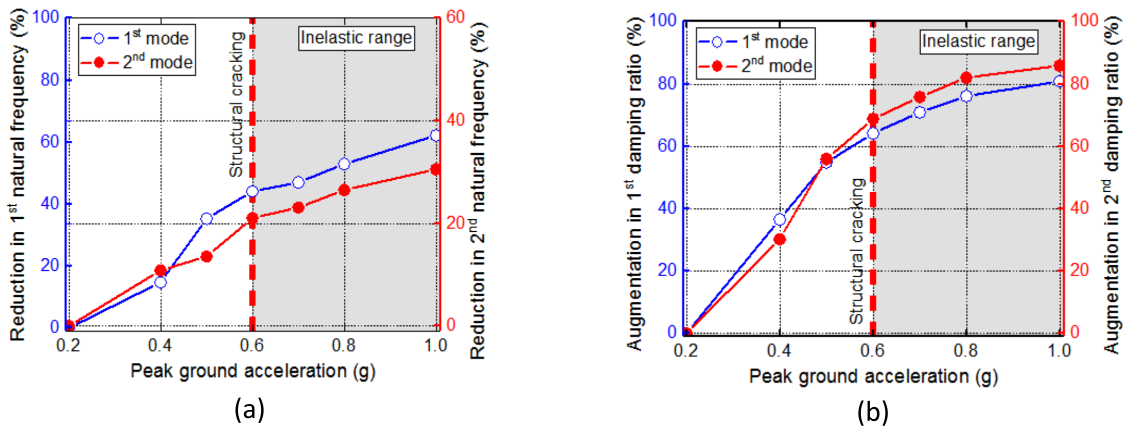
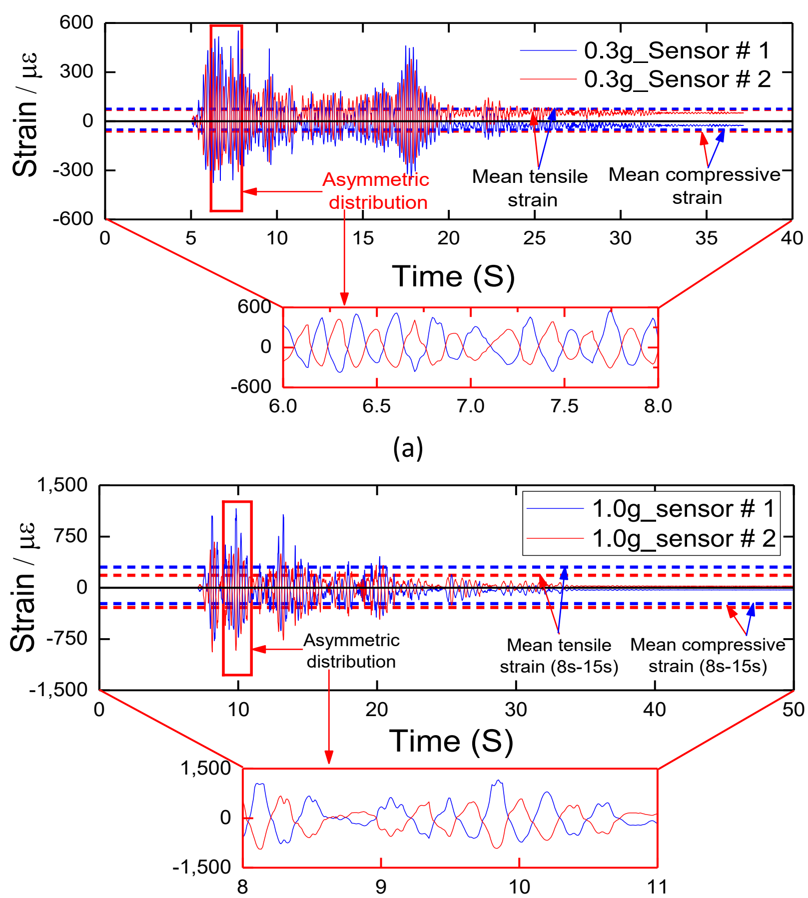


Figure 2: Dynamic characteristics (a). Natural frequency (b). Damping ratio

4. DAMAGE CONCENTRATION AT THE FLEXIBLE EDGE

The internal local seismic response within CCFE is presented in Figure 3. The FBG sensors namely sensor # 1 and sensor # 2 with their locations present in the same column but with different reinforcements, have been selected to analyse the influence of structural asymmetry on flexible edge response. Figure 3 shows the strain time-history responses of the selected sensors for the input ground motions of 0.3g and 1.0g in the elastic and inelastic states respectively.



(b)

Figure 3: Elastic and inelastic strain response at flexible edge under progressive seismic loading (a). PGA = 0.3 (b). PGA = 1.0g

Damage in CCFE under progressive seismic excitations is evident from the stiffness reduction response of the structural component presented in Figure 4.

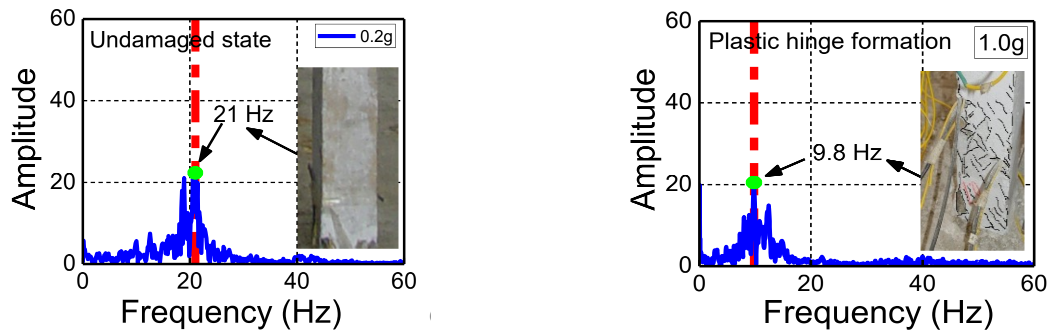


Figure 4: Local frequency shift: a measure of damage identification in CCFE

5. INFLUENCE OF STRUCTURAL ASYMMETRY ON LOCAL DAMAGE RESPONSE

The phenomenon of opposite trend of residual strains obtained from the sensors that were installed in the same column but with different reinforcement bars was due to the involvement of asymmetry in the structure and is demonstrated in Figure 5.

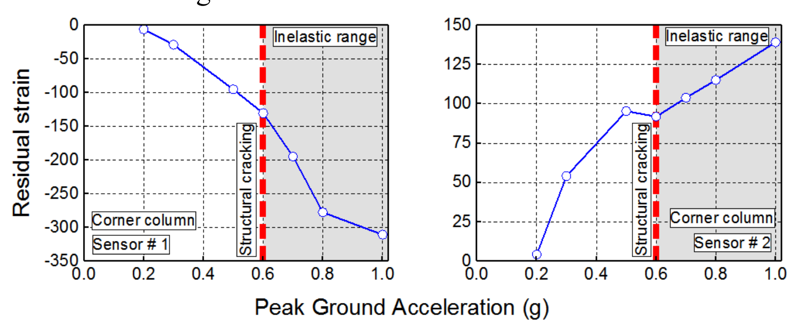


Figure 5: Residual strain under progressive loading corresponding to sensor # 1 and sensor # 2

6. FINITE ELEMENT MODELLING OF THE STRUCTURE

For further evaluation of the experimental response, a finite element model of the plan-asymmetric structure was developed in ABAQUS (Figure 6). The material properties for structural elements were used in accordance with the experimental data in the form of the engineering stress (σ_E) and engineering strain (ε_E). For this purpose, engineering stress and logarithmic strain relationship was used to obtain the true stress and true plastic strains as expressed in equation 1 and 2.

$$\sigma_T = \sigma_E(1 + \varepsilon_E) \tag{1}$$

$$\varepsilon_T^{pl} = \ln(1 + \varepsilon_E) - \frac{\sigma_T}{E} \tag{2}$$

Where E is the elastic modulus of concrete. In this research, equation 3 has been used to convert the total strains into in-elastic strains:

$$\varepsilon_{in}^{c} = \varepsilon_c + \varepsilon_{el}^{c} \tag{3}$$

Where ε_c is the total compressive strain, ε_{in}^{c} is the in-elastic compressive strain of concrete and ε_{el}^{c} is the elastic compressive strain of the undamaged concrete and equals the ratio of elastic compressive stress to the initial elastic modulus of concrete. In this work, the compressive stress-strain curve was utilized from Hsu and Hsu (1994).

Modulus of elasticity (E), cracking strain (ε_{cr}) and true stress (σ_T) were used for the considered grade of concrete to develop this model where cracking strain (ε_{cr}) can be calculated from the total strain using equation 4:

$$\varepsilon_{ck.}^{ten.} = \varepsilon_{tot.}^{ten.} + \varepsilon_{el.}^{ten.} \tag{4}$$

Where $\varepsilon_{ck.}^{ten.}$ is cracking tensile strain, $\varepsilon_{tot.}^{ten.}$ is the total tensile strain and $\varepsilon_{el.}^{ten.}$ is the elastic tensile strain of the undamaged concrete material.

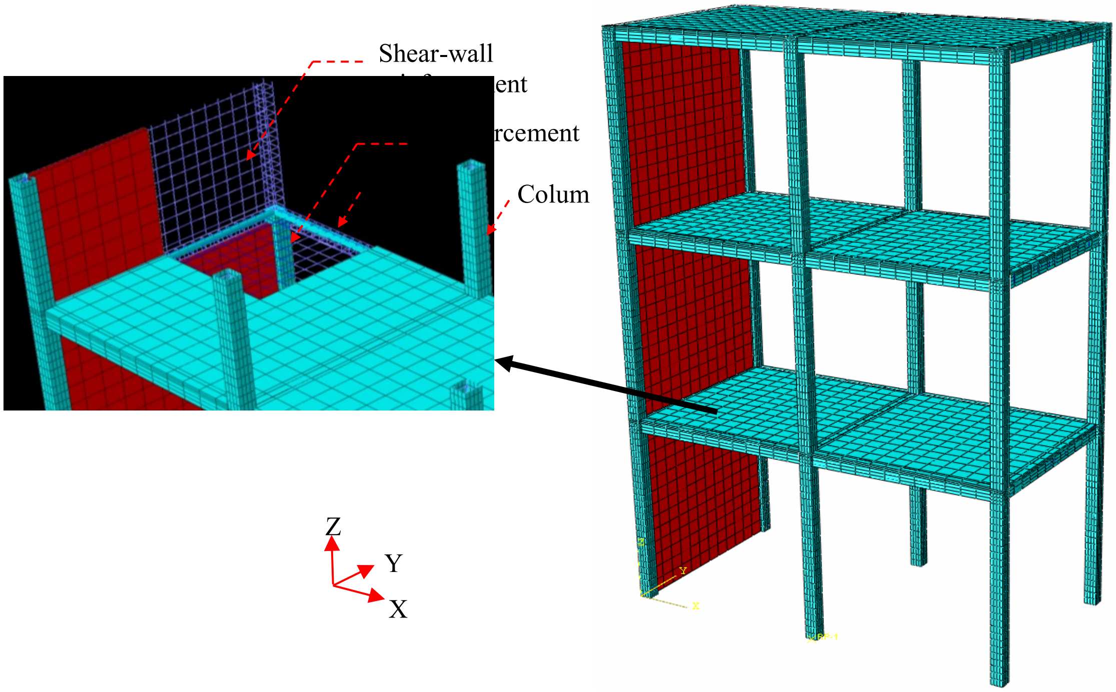
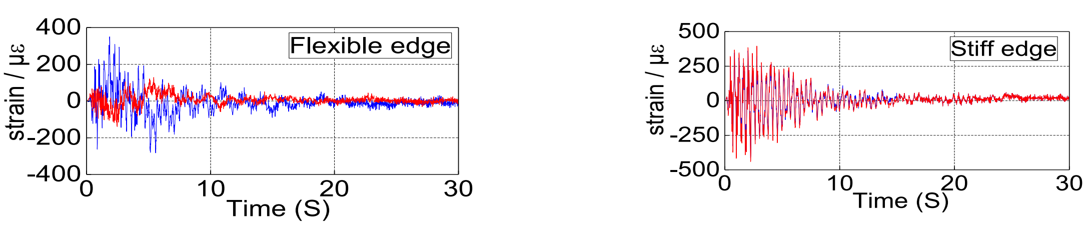


Figure 6: Finite Element Model of the plan-asymmetric structure

7. SIMULATED DAMAGE RESPONSE VARIATION AT FLEXIBLE AND STIFF EDGES

Local seismic responses in the FE model were observed at other critical locations of the vertical element at the flexible edge of the plan-asymmetric structure and were compared with the stiff edge response. No variation in compressive and tensile deformations is observed at the stiff edge as illustrated in Figure 7.



(a) (b)

Figure 7: Local response in the corner column at the flexible and stiff edge of the structure

8. CONCLUSIONS

Based on the experimental and numerical investigations, following conclusions are established:

- Corner columns at the flexible edge of the asymmetric structures are likely to remain partly under tension and partly under compression in terms of stored deformations. Conversely, compared with the flexible edge, corner columns at stiff edge of the structure may not experience this varying trend in the local response. For practical design applications, tensile deformations are an important consideration for potential uplift, opening up of the stirrups and buckling in the longitudinal reinforcement.
- Local frequency drop in CCFE can be used as a measure to develop early warning signs for the stiffness degradation of structural components. The discussed critical region of the plan-asymmetric structure is the most affected region to receive seismic damage under stiffness eccentricities.
- This study is helpful in understanding the local damage behaviour and further development of a practical engineering problem on local collapse mechanism in the vertical corner elements (Lu et al., 2016, Pearson and Delatte, 2005).

REFERENCES

Alam, Z., Zhang, C. & Samali, B. 2020. The role of viscoelastic damping on retrofitting seismic performance of asymmetric reinforced concrete structures. *Earthquake Engineering and Engineering Vibration*, 19(1), pp.223-237.

Alam, Z. & Zhang, C. 2019a. Influence of Earthquake Direction on Nonlinear Seismic Response of Plan-Asymmetric Structure. Paper presented at: *1st Conference on Sustainability in Civil Engineering*, August 01, 2019, Capital University of Science and Technology, Islamabad, Pakistan.

Alam, Z. & Zhang, C. 2019b. Seismic Performance of Multi-Storey Torsionally-Unbalanced Torsionally-Stiff (TU-TS) Structures. Paper presented at: *1st Conference on Sustainability in Civil Engineering*, August 01, 2019, Capital University of Science and Technology, Islamabad, Pakistan.2019.

Alam, Z. & Zhang, C. 2019c. Seismic Performance of Multi-Storey Torsionally-Unbalanced Torsionally-Flexible (TU-TF) Structures. Paper presented at: *1st Conference on Sustainability in Civil Engineering*, August 01, 2019, Capital University of Science and Technology, Islamabad, Pakistan.

Alam, Z., Zhang, C. & Samali, B. 2016. Response uncertainty under varying orientations of ground motions. *Mechanics of Structures and Materials XXIV.* CRC Press.

Ghorbanirenani, I., Tremblay, R., Léger, P. & Leclerc, M. 2011. Shake table testing of slender RC shear walls subjected to eastern North America seismic ground motions. *Journal of Structural Engineering,* 138**,** 1515-1529.

Hejal, R. & Chopra, A. K. 1989. Earthquake response of torsionally coupled, frame buildings. *Journal of Structural Engineering,* 115**,** 834-851.

Hsu, L. & Hsu, C.-T. 1994. Complete stress—strain behaviour of high-strength concrete under compression. *Magazine of Concrete Research,* 46**,** 301-312.

Kersey, A. D., Davis, M. A., Patrick, H. J., Leblanc, M., Koo, K., Askins, C., Putnam, M. & Friebele, E. J. 1997. Fiber grating sensors. *Journal of Lightwave Technology,* 15**,** 1442-1463.

Li, H.-N., Li, D.-S. & Song, G.-B. 2004a. Recent applications of fiber optic sensors to health monitoring in civil engineering. *Engineering Structures,* 26**,** 1647-1657.

Li, H. N., Sun, L. & Song, G. 2004b. Modal combination method for earthquake-resistant design of tall structures to multidimensional excitations. *The Structural Design of Tall and Special Buildings,* 13**,** 245-263.

Lu, Z., Chen, X., Lu, X. & Yang, Z. 2016. Shaking table test and numerical simulation of an RC frame-core tube structure for earthquake-induced collapse. *Earthquake Engineering and Structural Dynamics,* 45**,** 1537-1556.

Martinelli, P. & Filippou, F. C. 2009. Simulation of the shaking table test of a seven-story shear wall building. *Earthquake Engineering and Structural Dynamics,* 38**,** 587-607.

Pearson, C. & Delatte, N. 2005. Ronan point apartment tower collapse and its effect on building codes. *Journal of Performance of Constructed Facilities,* 19**,** 172-177.

Takewaki, I. 2005. A comprehensive review of seismic critical excitation methods for robust design. *Advances in Structural Engineering,* 8**,** 349-363.

Takewaki, I., Murakami, S., Fujita, K., Yoshitomi, S. & Tsuji, M. 2011. The 2011 off the Pacific coast of Tohoku earthquake and response of high-rise buildings under long-period ground motions. *Soil Dynamics and Earthquake Engineering,* 31**,** 1511-1528.

Zhang, C., Alam, Z. & Samali, B. 2016. Evaluating contradictory relationship between floor rotation and torsional irregularity coefficient under varying orientations of ground motion. *Earthquakes and Structures,* 11**,** 1027-1041.

Zhang, C., Alam, Z., Sun, L., Su, Z. & Samali, B. 2018. Fibre Bragg grating sensor-based damage response monitoring of an asymmetric reinforced concrete shear wall structure subjected to progressive seismic loads. *Structural Control and Health Monitoring***,** e2307.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Experimental Research on Post-Fire Repair of RC T-Beams Using Method of Curing

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Abstract
This paper presents experimental results of the structural performance of fire-damaged reinforced concrete (RC) T-beams subsequently repaired using the technique of post-fire curing. A total of six specimens were cast, of which two were controlled specimens, two specimens were exposed to ASTM standard fire and two specimens were re-cured after exposure to fire. The thermocouples embedded inside the beams provided valuable information about the heat transfer inside the beams. Experimental results showed that T-beams had considerably loss strength and stiffness when exposed to elevated temperatures. However, the beams regained most of their strength when re-cured by water. All of the beams exhibit failure in flexure during the third-point loading test.

Keywords
T-Beams, Post-Fire Repair, Curing

1. Introduction

Fire is one of the most destructive loads that can accidentally occur in a structure during its lifetime often when least expected. Although concrete structures have better fire performance as compared to steel and timber structures, its mechanical properties such as strength, stiffness and elastic modulus decreases with increase in temperature (Tufail *et al.*, 2016). The strength degradation is mainly caused by the difference in thermal expansion of the concrete constituents (Yaqub, 2010).

Various researcher have reported fire-damaged concrete can be repaired using the method of post-fire-curing by rehydrating the dehydrated products exposed to high temperature (Lin, Lin and Powers-Couche, 1996; Poon *et al.*, 2001; Chromá *et al.*, 2011). Shui *et al.*, (2009) reported that the dehydration of C-S-H gel in concrete occurs at 400°C exposed temperature. Several factors have been found responsible to influence the post-fire recovery of concrete after curing. Poon *et al.*, (2001) studied the strength and durability recovery of fire-damaged on the basis of types of curing conditions. Poon *et al.*, (2001) found that a significant recovery in strength can be achieved after curing the concrete for 28 days. The curing conditions are also proud to decrease porosity and pore sizes and reducing/closing cracks in concrete. This phenomenon is also confirmed by Henry, Darma and Sugiyama, (2014) by using the microstructure analysis using the technique of X-ray computer tomography (CT). In their study it was also observed that for recovering the concrete strength, moist environment is helpful, same reported by de Souza and Moreno Jr, (2010). Lin *et al.*, (2011) in their research studied the residual strength and ultrasonic pulse velocity (UPV) of fire-damaged concrete after curing the samples in air and water. Yaragal, Kittur and Narayan, (2015) studied the recovery of fire-damaged concrete after post-fire-curing and developed a model to predict the compressive strength of re-cured concrete. Park, Yim and Kwak, (2015) studied the post-fire tensile strength of concrete after being cured in conditions with different relative humidity, and it was directed that the residual tensile strength of fire-damaged concrete after post-fire-curing can be predicted by the hysteretic non-linearity parameter of the concrete.

Previous experimental studies on reinforced concrete beams include both simply supported (Lin, Gustaferro and Abrams, 1981; Lin, Ellingwood and Piet, 1988; Dwaikat and Kodur, 2009; Quanfeng *et al.*, 2013) and continuous beams (Xu *et al.*, 2012). The results from these tests indicate that the fire scenarios and depth to span ratio have more response towards fire in case of beams. A very little work has been carried out on post-fire repair of reinforced concrete beams. In this research, an experimental study is conducted on an economical repair of RC T-beams exposed to standard fire (ASTM E119, 2008).

2. Experimental Program

The experimental program of this study includes casting and testing of reduced scale T-beams in a purpose-built furnace to evaluate their fire performance and post-fire recovery of strength and stiffness.

2.1 Specimen Preparation

Six reinforced T-beams were cast to investigate the effects of post-fire curing on structural behavior of concrete. Each of the T-beam was 6.5 ft. long, and the web was 6 in wide × 3 in. deep, and flange 16 in wide × 3 in. deep. Each T-beam was reinforced with two bars of 3/8 in. diameter on the top side and two bars of 3/8-inch diameter on the bottom side. The transverse shear bars were 2/8-inch diameter bars at 6-inch centers designed to prevent shear failure. Geometry and reinforcement detailing are illustrated in Fig. 01. Two of the beams named as B_{O1} and B_{O2} were set as controlled beams, B_{D1} and B_{D2} were fire damaged beams whereas B_{R1} and B_{R2} beams are post fire damaged repaired beams. Table 01, provides details of the nomenclature of the beams.

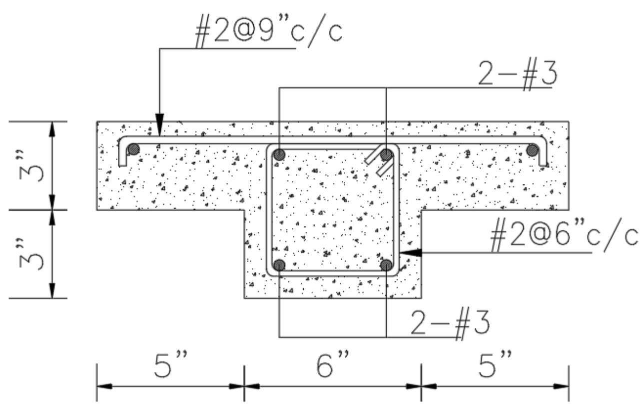

Figure 1: Geometry and reinforcement detailing of the beams

Table 01: Nomenclature and details of specimen

Beam	Nomenclature	Maximum Temp.	Fire Duration

Controlled Beams	B_{O1} and B_{O2}	20°C	-
Fire Damaged Beams	B_{D1} and B_{D2}	900°C	60 minutes
Post Fire Damaged Repaired Beams	B_{R1} and B_{R2}	900°C	60 minutes

2.2 Material Properties

Ordinary Portland Cement was used for casting of T-beams. Local sand with fineness modulus (FM) of 2.6 and coarse aggregate with a maximum size of ½-inch was used. Table 02, shows the details of mix proportions of concrete used in the casting of T-beams. The 28 days compressive strength of concrete cylinders tested as per ASTM C-39 standard method gave the average value of 4000 psi. The yield strength of reinforced bars was found to be 60,000 psi.

Table 02: Mix Proportion of Concrete

Property	Water	Cement	Sand	Aggregate
Mix (by weight)	0.55	1	1.5	3
Mix (lb./ft3)	11.5	25	37	75

2.3 Fire Testing in Furnace

Four of the six beams were exposed to high temperature in a custom-built furnace in such a way that the average temperature of the furnace follows ASTM E119 standard fire curve. Standard fire was achieved after several mock tests in a specially designed furnace for the purpose of beam testing at UET Taxila. The temperature inside the furnace was monitored using 15 thermocouples installed inside the furnace. Three Propane gas burners were used to produce high-intensity fire and uniform distribution of heat throughout the furnace. Fig. 02 shows furnace dimensions and test setup for beam tests in furnace (A: Ceramic protection on beam supports, B: Hinge support, C: Roller Support, D: Flange Section of the beam, E: Web Section of the beam, F: Fire from propane gas burners, G: Supports for beam, H: Top insulation of Furnace, & I: Furnace lining). Each beam was embedded with a network of thermocouples installed inside the beam-sections at different locations to determine the intensity of heat transfer inside the beam section. Four K-type beaded thermocouples were installed in each three sections of an individual beam that makes a total of 12 thermocouples in a single beam. Fig. 03 shows the detail of thermocouples in an individual beam.

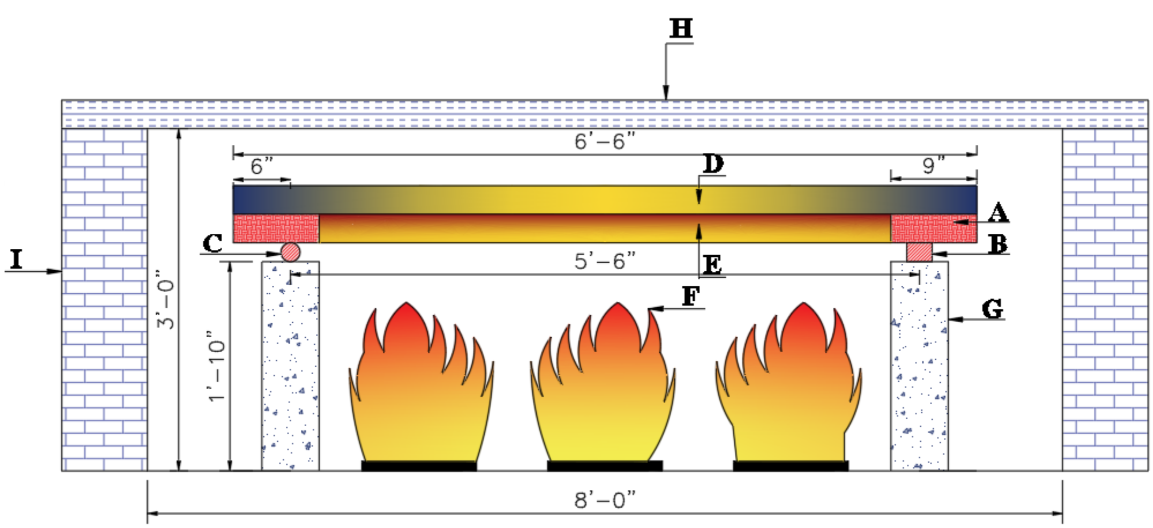

Figure 2: Fire testing of the beams inside the furnace

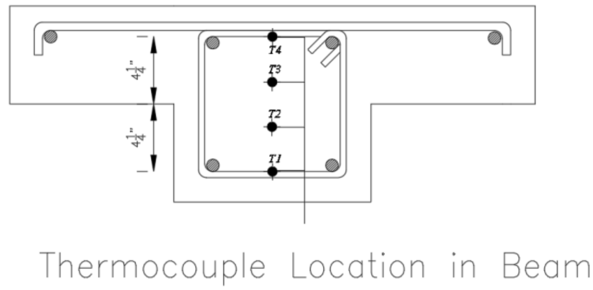

Figure 3: Thermocouples placement inside the beam sections

The maximum temperature of 900°C was maintained in the furnace however at some instances the temperature in furnace raised to 920°C as well which was controlled through close monitoring and adjusting the gas pressure. Spalling in flange portions of the beam was observed in all the beams. The web portion remained intact. The average fire curve achieved inside the furnace with comparison to standard ASTM E119 fire curve exhibited similarity. The core temperature of the beam raised slowly in the beginning but continue to raise even when the furnace gas supply was terminated. Fig. 04 shows a comparison of furnace fire with ASTM E119 fire, highlighting good agreement between furnace temperature and ASTM E119 curve. The temperature variation throughout the sections of beams (T1: Thermocouple attached to the bottom reinforcement of the beam section, T2: Thermocouple attached to the bottom-center of the beam section, T3: Thermocouple attached to the top-center of the beam section, & T4: Thermocouple attached to the top reinforcement of the beam section) is shown in Fig. 05. Overall, the beams did not show any sign of major cracking or collapse. However, the surface was porous the concrete surface color was changed.

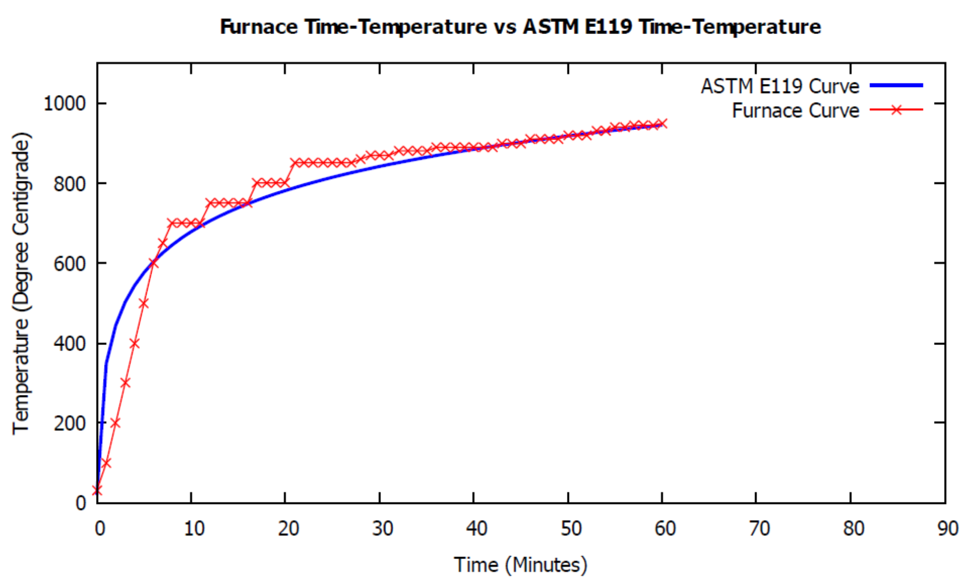

Figure 4: Comparison of Average Furnace Time-Temperature with ASTM E119 Time-Temperature Curve

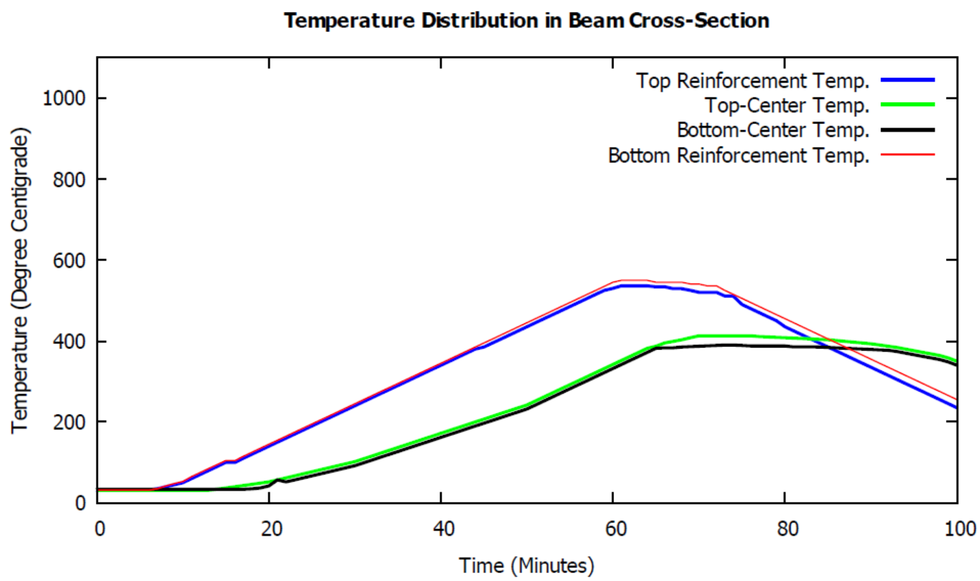

Figure 5: Temperature measured by thermocouples in the beam cross-section

2.4 Post-Fire Curing

After one week of the fire tests, two of the four heated beams were submerged into a water tank for post-fire curing. The beams remained in the water for 28 days. After 28 days, the beams were placed at room temperature for one week before testing.

2.5 Load Tests

The flexural strength of the beams was investigated using ASTM third point loading test (ASTM C78, 2010). The load was applied by hydraulic jack as shown in Fig. 06 and two LVDTs were placed below the beam to calculate the deflection at required load. The load was measured by a calibrated load cell with maximum capacity of 2000 KN. The application points of load on the beam were spaced at distance of L/3 from each support. The supports were placed distance of 6” from each end of the beam. The cracking pattern and displacements were measured at each interval of the applied load.

Figure 6: Temperature measured by thermocouples in the beam cross-section

3. Test Results and Discussion

At the beginning of the load test, the deflection at the mid of controlled beams increased linearly with the applied force. The first cracks were produced at the web portion in the middle of beams B_{O1} and B_{O2} when the applied loads reached to 26.88 kN and 27.54 kN respectively. These loads show the introduction of non-linearity in the beams. After that, the cracks kept on increasing with the applied load until the loads reached to 44.8 kN and 43.5 kN for B_{O1} and B_{O2} respectively with cracks reaching to the flange portion of the beams. Upon unloading, the beams showed inelastic behavior and permanent deformation of 8.66 mm and 9.01 mm was observed when the load was completely removed. The load-deformation curve for controlled samples is shown in Fig. 07.

The fire-damaged beam showed a prominent decrease in residual strength and stiffness. The first crack in the beams was introduced with applied load reaching 8.96 kN and 9.5 kN and the maximum load-carrying capacity was observed as 27 kN & 29 kN for beams B_{D1} and B_{D2}. A considerable amount of deformation up to 22 mm at a very small load shows the degradation of stiffness in the beams. Fig. 08 shows a loss of 60.17% strength when the beams were exposed to 900°C fire for the time of 1 hour.

2 of the 6 beams named as B_{R1} and B_{R2} were submerged in curing tank for 28 days after exposing them to 900°C for 1 hour. The beams performed very well during load tests and showed a considerable recovery in strength and stiffness. The beams performed linearly up to the load of 25.5 kN and 24.9 kN whereas the maximum load-carrying capacity for the beams B_{R1} and B_{R2} were observed as 40 kN and 39 kN respectively. The beams showed the recovery of 90% strength when repaired with curing after exposure to fire shown in Fig. 08. This is most probably due to repair of micro-cracks in concrete.

All the beams performed good in shear and no shear failure was observed in any beam. Flexure failure was prominent in all cases and no rupture of steel reinforcement was observed.

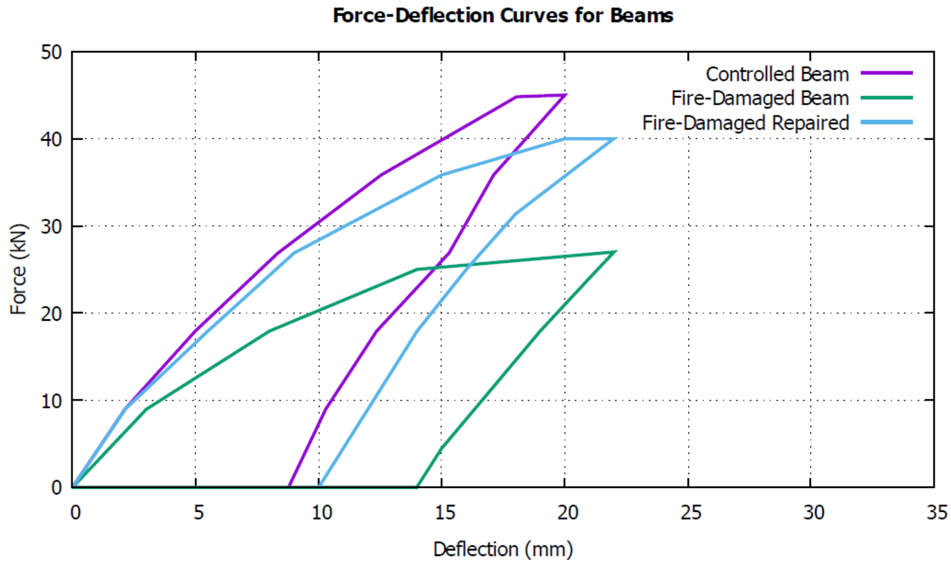

Figure 7: Comparison of force-deflection behavior of beams

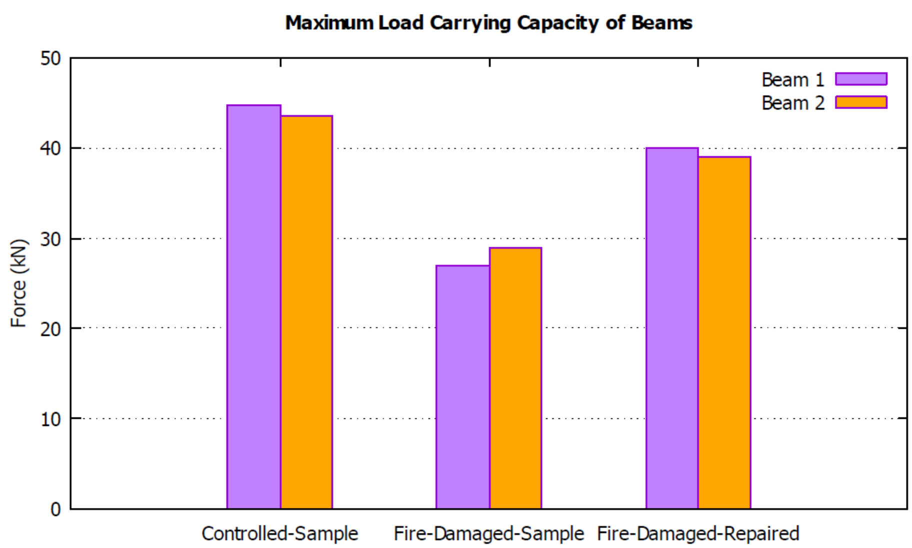

Figure 8: Maximum load-carrying capacity of beams

4. Conclusions

In this paper, experimental results of the structural performance of fire-damaged reinforced concrete (RC) T-beams subsequently repaired using the technique of post-fire curing. A total of six specimens were cast, of which two were controlled specimens, two specimens were exposed to ASTM standard fire and two specimens were re-cured after exposure to fire. The following conclusions are drawn from this experimental work:

i. The under-reinforced concrete T-beams not exposed to fire behave in flexure manner which is very typical in case of T-beams.

ii. T-beams degrade in strength and stiffness considerably when exposed to high-intensity fire for a duration of 1 hour. The residual strength of the fire-damaged beam is 40% when compared with controlled undamaged beams. The beams loss up to 60% of the strength.

iii. The strength and stiffness can be recovered to considerable extent by submerging the beams in water. This is due to repair of the microcracks in concrete that were produced to due exposure of high intensity fire. The post-fire water cured beams recovered 89% of its original strength after being cured for 28 days in curing tank.

5. Acknowledgment

This work is financially supported by Higher Education Commission of Pakistan under National Research Program for Universities (NRPU) Grant No. 10607.

6. References

Alonso, C. and Fernandez, L. (2004) 'Dehydration and rehydration processes of cement paste exposed to high temperature environments', *Journal of Materials Science*, 39(9), pp. 3015–3024. doi: 10.1023/B:JMSC.0000025827.65956.18.

ASTM C78 (2010) 'Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) 1', in *Hand The*, pp. 1–4. doi: 10.1520/C0078.

ASTM E119 (2008) 'Standard test methods for fire tests of building construction and materials', 552(C), pp. 1–33. doi: 10.1520/E0119-14.

Chromá, M. *et al.* (2011) 'Concrete rehydration after heating to temperatures of up to 1,200 C', in *Proc., 12th Int. Conf. on Durability of Building Materials and Components*.

Dwaikat, M. B. and Kodur, V. K. R. (2009) 'Response of Restrained Concrete Beams under Design Fire Exposure', *Journal of Structural Engineering*, 135(11), pp. 1408–1417. doi: 10.1061/(asce)st.1943-541x.0000058.

Henry, M., Darma, I. S. and Sugiyama, T. (2014) 'Analysis of the effect of heating and re-curing on the microstructure of high-strength concrete using X-ray CT', *Construction and Building Materials*. Elsevier Ltd, 67, pp. 37–46. doi: 10.1016/j.conbuildmat.2013.11.007.

Lin, T. D., Ellingwood, B. and Piet, O. (1988) *Flexural and shear behavior of reinforced concrete beams during fire tests*. Washington, D.C.

Lin, T. D., Gustaferro, A. H. and Abrams, M. S. (1981) *Fire endurance of continuous reinforced concrete beams*. Skokie, 111.

Lin, W.-M., Lin, T. D. and Powers-Couche, L. J. (1996) 'Microstructures of fire-damaged concrete', *Materials Journal*, 93(3), pp. 199–205.

Lin, Y. *et al.* (2011) 'The effect of post-fire-curing on strength velocity relationship for nondestructive assessment of fire-damaged concrete strength', *Fire Safety Journal*. Elsevier, 46(4), pp. 178–185. doi: 10.1016/j.firesaf.2011.01.006.

Park, S. J., Yim, H. J. and Kwak, H. G. (2015) 'Effects of post-fire curing conditions on the restoration of material properties of fire-damaged concrete', *Construction and Building Materials*. Elsevier Ltd, 99, pp. 90–98. doi: 10.1016/j.conbuildmat.2015.09.015.

Poon, C.-S. *et al.* (2001) 'Strength and durability recovery of fire-damaged concrete after post-fire-curing', *Cement and Concrete Research*, 31(9), pp. 1307–1318. doi: 10.1016/S0008-8846(01)00582-8.

Quanfeng, W. *et al.* (2013) 'Experimental research on fire-resistant performance of HRBF 500 reinforced concrete simple beams', *Building Structure*, 1.

Shui, Z. *et al.* (2009) 'Cementitious characteristics of hydrated cement paste subjected to various dehydration temperatures', *Construction and building materials*. Elsevier, 23(1), pp. 531–537.

de Souza, A. A. A. and Moreno Jr, A. L. (2010) 'Assessment of the influence of the type of aggregates and rehydration on concrete submitted to high temperatures', *Revista IBRACON de Estruturas e Materiais*. SciELO Brasil, 3(4), pp. 477–493.

Tufail, M. *et al.* (2016) 'Effect of Elevated Temperature on Mechanical Properties of Limestone, Quartzite and Granite Concrete', *International Journal of Concrete Structures and Materials*. Springer Netherlands, 10(1), pp. 1–6. doi: 10.1007/s40069-016-0175-2.
Xu, Y. *et al.* (2012) 'Experimental Study on Residual Flexural Behavior of Reinforced Concrete Beams after Exposure to Fire', 458, pp. 183–187. doi: 10.4028/www.scientific.net/AMR.457-458.183.
Yaqub, M. (2010) *Axial compressive and seismic shear performance of post-heated columns repaired with composite materials Doctor of Philosophy 2010*. The University of Manchester. Available at: https://www.escholar.manchester.ac.uk/uk-ac-man-scw:101698.
Yaragal, S., Kittur, M. and Narayan, K. (2015) 'Recuring Studies on Concretes Subjected to Elevated Temperatures and Suddenly Cooled by Water Quenching', *Journal of Structural Fire Engineering*, 6(1), pp. 67–76. doi: 10.1260/2040-2317.6.1.67.

11th International Civil Engineering Conference (ICEC-2020)
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March 13-14, 2020, Karachi, Pakistan.

Behavior of interlocking plastic-block wall with opening under harmonic loading using locally developed shake table

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Abstract

Earthquake is one of the dangerous and life-threatening natural disaster. The impacts of the vibrations generated by earthquakes normally prompts the destruction of civil engineering structures such as buildings, bridges, and dams etc. Specifically, masonry buildings in seismic zones of rural and urban regions throughout the world constitutes a hazard to human life. In developing countries, earthquake resistant and economical housing in the earthquake prone areas is the demand of time. Interlocking block structures have emerged as a new construction technique for earthquake resistant houses. Researchers have investigated many mortar-free interlocking techniques. But interlocking plastic-block structures are still not explored. To start with, prototype interlocking plastic-blocks wall having opening in the form of window is considered for making the mortar-free structure. In this study, behavior of prototype interlocking plastic-block wall is examined against harmonic loading using locally developed low-cost shake table. Wall consists of forty two plastic blocks having window opening in the middle and bottom blocks layer fixed with the shake table. Two accelerometers are used: one is attached at the shake table to record the base excitation and one is attached at the top block to record the wall response. The behavior of structure in terms of acceleration-time, velocity-time and displacement-time histories is recorded. Energy absorption of the structure is determined. Empirical equations are developed keeping in mind the geometry of interlocking blocks, wall height and input loading parameters. This study determines the future directions for exploring the in-depth behavior of interlocking plastic-block structures.

Keywords

Interlocking plastic-block, Accelerometer, Harmonic loading, Wall with opening.

1. Introduction

Earthquake is sudden voilent shaking of the earth's surface. It occurs naturally and causes one of the hazardous and deadly catastrophe. Earthquakes produce different damaging impacts to the zones they act on. This incorporates harm to structures and in worst scenarios the loss of human life. The impacts of the vibrations generated by earthquakes normally prompts the destruction of civil engineering structures such as buildings, bridges, and dams etc. Specifically, masonry buildings in seismic zones of rural and urban regions throughout the world constitutes a hazard to human life. Because strong ground motions generated by earthquake badly damage the masonry structures. Ground acceleration is transferred from ground to structure foundation which causes shearing of masonry walls due to inertia.The Kashmir earthquake of October, 2005 caused more than 86,000 causalities, more than 80,000 human injuries and an estimated total economic loss of $5.2 billion (Mulvey et al . 2008). Sichuan earthquake in 2008, having magnitude of 8.0 caused 70,000 casualties, 216,000 structural failures, including 6890 school structures (Zhang and Jin 2008). In Nepal earthquake of 2015, 0.15 million people were displaced due to severe structural damages in the affected region (Chen et al. 2016).

The primary reason behind the destruction of masonry buildings either partial or full, is usage of conventional unconfined masonry technique. In addition, because of design deficiencies the majority of the brick masonry buildings face severe damages during earthquakes. In 2010 Haiti earthquake, 80% to 90% of the masonry structures were declared partially or fully damaged by the Haiti government (Desroches et al. 2011). In Gorkha earthquake, number of 0.5 million masonry buildings were entirely collapsed and other 0.2 million were partially damaged (Gautam and Chaulagain 2016). During the quakes of 2010 and 2011 in Canterbury, 72% of the identified walls were damaged due to out of the plane damages and 28% were due to in plane damages (Giaretton et al. 2016). Recently in 2018, Lombok earthquake of Indonesia damaged more than 1000 houses (Elvin and Uzoegbo 2018).

Due to absence of earthquake resistant construction techniques, these countries grieve from huge human loss during strong ground motion. The literature indicates that, various construction techniques in the form of structural components for the construction of earthquake resistant masonry buildings have been adopted. For example, provision of vertical stiffeners and lintel beams in the masonry walls. Similarly, Ali (2013) developed mortar free interlocking block structure as a new construction technique for earthquake resistant houses and reported energy dissipation due to comparative movement at the interlocking block edges. Coconut fiber reinforced interlocking mortar-free block with post-tensioned coconut fiber ropes were tested against dynamic loading (Ali et al. 2017).

To the best of author knowledge, no study has been conducted to investigate the behaviour of interlocking plastic-block wall with opening under harmonic loading using locally developed shake table. The main purpose is to explore the potential of interlocking plastic-blocks for earthquake resistant housing. To start with, smallscale interlocking wall with opening is considered. For real application, full-scale plastic blocks would be needed along with some mechanism for wall connections with foundation and diaphragm. However, this would be tackled if favourable results are obtained from small-scale testing. Waste plastic can be recycled for useful interlocking plastic-blocks. Also, fire-resistant paints may be needed which is outside the scope of current work. In this study, the behavior of plastic block wall with opening in terms of acceleration-time, velocity-time and displacement-time histories is recorded. Energy absorption of interlocking plastic-block wall is experimentally investigated. Empirical equations are also being developed from the obtained results.

2. Experimental Procedures

2.1 Interlocking plastic block wall with opening on shake table and its instrumentation with background

Khan and Ali (2018) proposed interlocking plastic-blocks for earthquake-resistant construction and performed dynamic experimental techniques on prototype. In the study, the base dimensions of proposed block for real construction was provided as 150 mm x 150 mm with a total height of 140 mm including 30 mm interlocking key height. On the other hand, prototype base dimensions was 62 mm x 62mm with a total height of 53 mm including 12 mm interlocking key height. And prototype plastic block having 25g mass was used. To study dynamic performance of interlocking plastic block prototype column, locally developed low cost shake table was used. Simple 1D shake table was prepared by using local human resource and materials.

Current study is continuation of Khan and Ali (2018) work, and focuses on behavior of interlocking plastic block wall with opening. Wall with opening consists of fourty two interlocking plastic-blocks (n=42), making a total height (H) of 328 mm. It is having an opening in form of window in the middle. The dimmensions of opening is 124mm x 82mm. Wooden lintel band is provided above the opening for support mechanism. In addition, rubber band are tied up from bottom to top through mid of blocks to provide vertical stiffeness in the wall. Fixed base with the help base plates and nut bolts is provided. No mass is provided at the wall top. However, the total mass of wall (M) is 1.295 Kg. The instrumentation of wall placed on shake table is shown in Figure 3. Schematic diagram is shown in Figure 3a and test set up is shown in Figure 3b. Two accelerometers are used: one at shake table to record ground motion and other at wall top to record the wall response. Accelerometers are connected to computer system and data from accelerometers to computer is transferred using two types of software such as arduino and visual studio. Wall response in out-of-plane direction in terms of acceleration-time is recorded and velocity-time and displacement-time histories are obtained using seismosignal software.

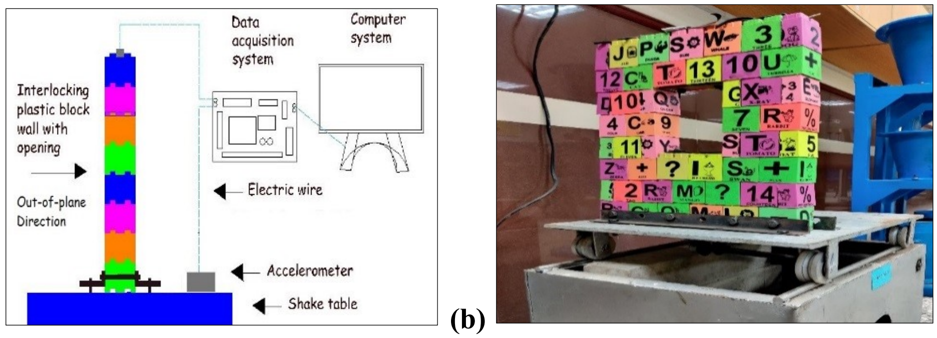
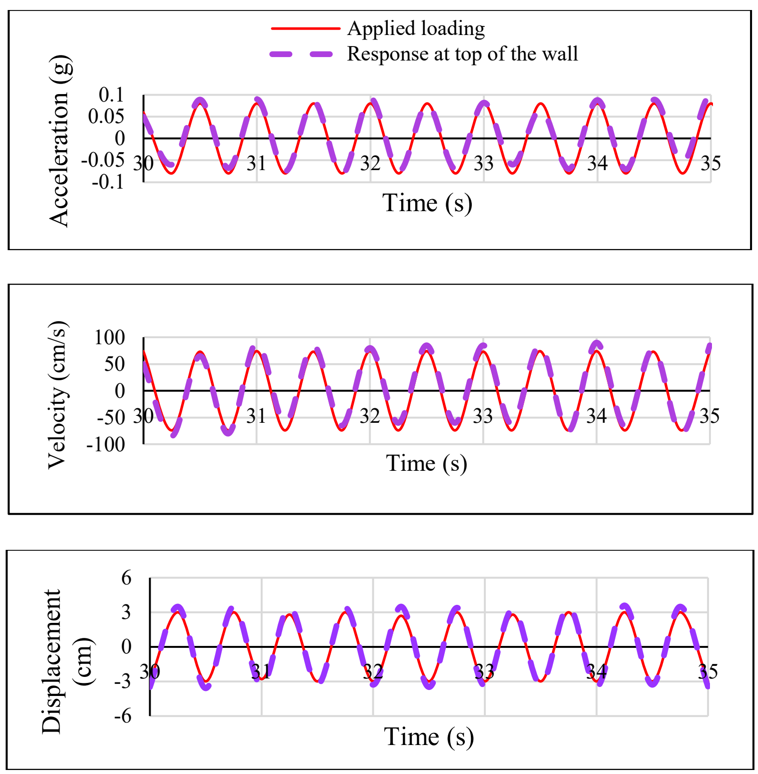


Figure 1: Arrangement of interlocking plastic block wall with opening in out-of-plane direction on shake table; (a) schematic diagram, (b) test setup

2.2 Behavior of prototype interlocking plastic-blocks

Prototype interlocking plastic-block wall with opening was tested on shake table under harmonic loading with amplitude of around ±4.5 cm and frequency of 2 Hz (i.e. time period of 0.5 s or rotation of 120 rpm).



a)

b)

c)

Figure 2: Behavior of interlocking plastic block wall with opening for intermediate 5 seconds; (a) acceleration – time history, (b) velocity – time history and (c) displacement – time history

3. Results

3.1 Response in terms of Acceleration – time, velocity – time, and displacement - time histories:

Reaction of interlocking plastic-block wall in form of acceleration – time history, velocity – time history and displacement – time history during the period from 30 s to 35 s is shown in Figure 2. The blue dash line shows response at top of wall while red full line shows applied loading. These histories are satisfactory to huge degree to examine the dynamic reaction of wall. As depicted before, acceleration-time history was recorded with the help of accelerometer and then it was being converted to velocity- time and displacement-time histories using software seismosignal.

As described earlier, locally developed shake table is only able to apply precise harmonic loading. There is a little variation exists in amplitude of different cycles. The averaged acceleration, velocity and displacement of base motion (i.e. $\ddot{u}_g$, $\mathring{u}_g$, *and* u_g, respectively) is taken as applied loading. These are 0.08 g, 75 cm/s and 3 cm, respectively. Similarly, the averaged acceleration, velocity and displacement at wall's top (i.e. $\ddot{u}_t$, $\mathring{u}_t$, *and* u_t, respectively) is taken as wall response. These are 0.085 g, 90 cm/s and 3.2 cm, respectively. It may be noted that the amplitude of each cycle is considered for taking the average of any particular parameter.

3.2 Energy absorption and base shear – displacement curves:

Its is assumed that the total mass of structure (M) is lumped at wall's top where its response acceleration time (i.e., $\ddot{u}_t$ - t) history is recorded. Base shear is calculated as M . $\ddot{u}_t$. Typical base shear - displacement curves for a single cycle are shown in Figure 5. This is calculated as per working of Ali et al. (2013). Table 1 shows the averaged energy absorption in one cycle as well as total energy absorbed. Area within the loop is taken as energy absorption. In seismic event, interlocking plastic-block wall with opening can absorb more energy, because of the relative movement at block interfaces. Energy conservation generally applies to uplift and rocking behaviors, it is typically the rocking impacts that could lead to energy absorption, but rocking/uplift works because it reduces secant stiffness of structure and hence detune the effect of earthquakes. Experimentation is being done with observation that energy dissipation is because of relative movement or uplift of block which will be studied in future.

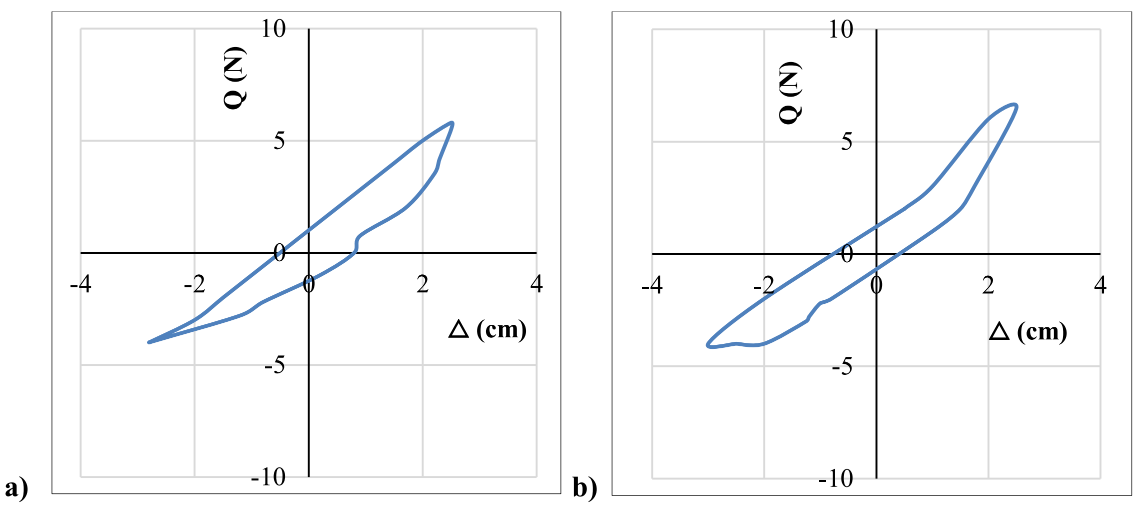


Figure 3: Base shear - displacement curves; (a) maximum loop area in positive direction, and (b) maximum loop area in negative direction

Table 1: Energy absorption during the harmonic loading

Structure	**Average energy absorbed in one cycle (Nm)**	**Total energy absorbed (Nm)**
Interlocking plastic-block wall	0.54	36.4

with opening

3.3 Development of empirical equations:

Khan and Ali (2018) developed empirical equations incorporating the geometry of interlocking blocks, column height, column response and input loading parameters. Following empirical equations are developed for predicting the wall response by incorporating further new variable for the wall:

$$\ddot{u}_t = \frac{\frac{a}{h^2}}{n} \mathrm{mK}^{\left(1+\frac{2n}{100}\right)} \ddot{u}_g$$

$$\mathring{u}_t = \frac{\frac{a}{h^2}}{n} \mathrm{mK}^{\left(1+\frac{2n}{100}\right)} \mathring{u}_g$$

$$u_t = \frac{\frac{a}{h^2}}{n} \mathrm{mK}^{\left(1+\frac{2n}{100}\right)} u_g$$

Where $\ddot{u}_g$, $\mathring{u}_g$, *and* u_g are averaged ground acceleration, velocity and displacement, respectively. Their corresponding values are 0.08 g, 75 cm/s and 4.5 cm, respectively. $\ddot{u}_t$, $\mathring{u}_t$, *and* u_t are response acceleration, velocity and displacement, respectively. a, h, n, and m are block base area, key height, total number of blocks, and number of blocks along the length of wall in a singke layer, respectively. Their corresponding values are 62 mm x 62 mm, 12 mm, 42, 41 mm * 8 + 12 mm = 340 mm, respectively. K is coefficient having dimensionless value of 0.5. In Table 2, comparison of experimental and empirical values of wall response is shown. It can be noted that experimental values ar good agreement with empirical values. The percentage difference is less than or equal to 7%.

Table 2: Energy absorption during the harmonic loading

Wall response	Experimental values	Empirical values	Percentage difference
Acceleration (g)	0.085	0.091	7%
Velocity (cm/s)	90	95.86	6.51%
Displacement (cm)	3.2	3.40	6.25%

4. Discussion

Application of harmonic loading using locally developed shake table is not much accurate with respect to different frequencies and fixed amplitude. However, it is able to produce precise harmonic loading to some extent so that the dynamic behavior of structure under observation can be studied. This is so because the applied harmonic loading is taken as the base ground motion and response of the structure is studied with respect to it. On the other hand, the observed behavior of interlocking plastic-block wall with opening is more or less same as that reported in other researches (Ali et al. 2013 and Elvin and Uzeogbo 2011). The studied wall has shown positive potential in form of structural stability and energy absorption. So, it should be explored in detail for wall in connection with other structural elements.

5. Conclusion

Following conclusions can be drawn from the conducted study:

- Response of wall in terms of averaged acceleration, velocity and displacement is amplified a little bit at its top compared to applied loading at base.

- Interlocking plastic-block uplifts during applied harmonic loading constitutes to energy absorption.
- Experimental behavior of wall is in good agreement with empirical values.
- The above outcome is favorable indicating the exploration of its in-depth behavior. Next step should be the dynamic behavior of interlocking plastic-block wall along with other structural members i.e., diaphragm etc. Numerical modelling on mechanism of energy dissipation and influence of reducing the mass on it is planned in parallel research.

6. Acknowledgement

The authors would like to thank Engr. Nakhshab Ajaz, Engr. Sohail Afzal and Engr. Fayyaz Khan who helped throughout the research work. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

7. References

Ali, M., Briet, R., and Chouw, N. (2013). "Dynamic response of mortar-free interlocking structures". *Construction and Building Materials*, vol. 42, pp. 168-189.

Ali, M. (2017). "Role of Post-tensioned Coconut-fibre Ropes in Mortar-free Interlocking Concrete Construction During Seismic Loadings". *KSCE Journal of Civil Engineering*, vol. 22, no. 4, pp. 1336-1343.

Chen, H., Xie, Q., Li, Z., Xue, W., and Liu, K. (2016). "Seismic Damage to Structures in the 2015 Nepal Earthquake Sequences". *Journal of Earthquake Engineering*, vol. 21, no. 4, pp. 551-578.

Desroches, R., Comerio, M., Eberhard, M., Mooney, W., and Rix, G. J. (2011). "Overview of the 2010 Haiti Earthquake". *Earthquake Spectra*, vol. 27, no. 1. pp. 124-130.

Elvin, A. and Uzoegbo, H.C. (2011). "Response of a full-scale dry-stack masonry structure subject to experimentally applied earthquake loading". *Journal of the South African Institution of Civil Engineers*, vol. 53, pp. 22 -32.

Khan, F., and Ali, M. (2018). "Behavior of interlocking plastic- block structure under harmonic loading using locally developed low-cost shake table". "Annual Australian Earthquake Engineering Society Conference". Nov: 16-18. Paper 51.

Gautam, D., and Chaulagain, H. (2016). "Structural performance and associated lessons to be learned from world earthquakes in Nepal after 25 April 2015 (MW 7.8) Gorkha earthquake". *Engineering Failure Analysis*, vol. 68, pp. 222-243.

Giaretton, M., Dizhur, D., Porto, F., and Ingham, J. M. (2016). "Construction Details and Observed Earthquake Performance of Unreinforced Clay Brick Masonry Cavity-wall". *Engineering Structures*, vol. 6, pp. 159-169.

Mulvey, J. M., Awan, S. U., Qadri, A. A., and Maqsood, M. A. (2008). "Profile of injuries arising from the 2005 Kashmir Earthquake: The first 72 h". *Injury Prevention*, vol. 39, pp. 554-560.

Zhang, M., and Jin, Y. (2008). "Building damage in Dujiangyan during Wenchuan Earthquake". *Earthquake Engineering and Engineering Vibration*, vol. 7, no. 3, pp. 263-269.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Analysis of Adhesion and Moisture Susceptibility of Different Modified Bitumen Using Bitumen Bond Strength and Rolling Bottle Testing Techniques

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Abstract
The strong adhesion between binder and aggregate is considered one of the main parameters for durable asphalt concrete. Due to the presence of moisture, the bond between bitumen and aggregate weakens and the failure of pavement takes place. There are several methods to mitigate the damaging effects of moisture on asphalt pavements, one of them is to use modifiers in asphalt concrete. In this study moisture damage and adhesion of binder (60-70, 80-100 pen bitumen) modified with wax, Hydrated Lime (HL), and Low-Density Polyethylene (LDPE) was assessed. The modifier was added according to the weight of the binder, 1% and 2% for each set of tests. For assessment of adhesion and moisture susceptibility, Bitumen Bond Strength (BBS) test using Pneumatic Adhesion Tensile Testing Instrument (PATTI) was used to calculate Pull Off Tensile Strength (POTS) and to observe the failure type after dry and wet conditions. The Rolling Bottle Test (RBT) was performed to quantify the moisture susceptibility of the asphalt mixture. Results of experimental investigation revealed that modified bitumen, by incorporating 2% each of the HL and the LDPE, showed higher bond strength and less moisture susceptibility as compared to the controlled bitumen whereas wax modified bitumen showed negative results.

Keywords
Moisture susceptibility, adhesion, asphalt concrete, bitumen bond strength test, rolling bottle test.

1. Introduction

Moisture damage is one of the main reasons behind the early failure of pavements. Due to moisture, water penetrates the bitumen- bitumen interlayer and bitumen-aggregate interface which weakens the bond. When the bond between bitumen-aggregate gets weak, it lowers the strength and stiffness of asphalt mixture which leads to the pavement failure. The failure is due to two main reasons: firstly, the loss of cohesion between bitumen-bitumen interlayer, known as softening and secondly, the loss of adhesion between bitumen-aggregate interface, known as stripping. Although all distresses are not caused by moisture directly, but its presence increases the harshness of already existent distresses such as cracking, potholes, rutting, and raveling. Moisture damage in pavement not only causes distresses but also enhances fatigue cracking.

Among the many factors that have an effect on adhesion and moisture damage of pavements, cohesive failure (which starts from the loss of stiffness of mixture) and adhesive failure (bitumen wipes off from the aggregate surface) are the most important. Repeated freeze-thaw cycles also accelerate the distresses in the pavement. Since the performance of asphalt pavement reduces due to the effect of moisture, a number of researchers put their prudent efforts to simulate moisture damage in the laboratory over the years. Asphalt technologists have carried out effective research to develop laboratory tests to differentiate between poor and good performing bituminous mixes. The tests were further divided into two main categories: one for compacted mixtures and other for loose mixtures (Solaimanian et al. 2003).

2. Problem Statement

The strong adhesion between binder and aggregate is the fundamental parameter for durable asphalt concrete. Due to the presence of moisture, the bitumen-aggregate bond gets weak and the pavement failure takes place. The research is focused on the effects of the wax, the HL and the LDPE on adhesion; and to enhance moisture damage resistance of different grades of bitumen by using them with the same aggregate.

3. Aims and objectives

The main objective of this research is to examine the adhesive bond strength between bitumen-aggregate interface and cohesive bond strength between bitumen-bitumen interlayer in loose coated asphalt mixtures with wax, the HL and the LDPE as modifiers. Adhesion and moisture susceptibility are assessed using the Bitumen Bond Strength Test and Rolling Bottle Test.

4. Literature review

There are many useful, popular and cost-effective ways used in the transportation industry to mitigate adhesion and moisture damage by using various modifiers and additives.
(Cihlářová et al. 2018) studied the effects of fatty acids of polyamides on the adhesion properties of asphalt. The results of the rolling bottle and indirect tensile tests show the improvement in adhesion between the bitumen-aggregate interface. (Liu et al. 2014) performed five different empirical tests to check the effect of moisture on the stiff and soft binders. The Rolling Bottle Test and boiling water test results show that stiff binders showed better water resistance as compared to the soft binders. (Ali et al. 2016) evaluated the fluidity and adhesion strength of bitumen modified with cement, HL, and nano-clay by using the Bitumen Bond Strength test. Bitumen modified by using steel substrate showed better results as compared to the controlled bitumen. (Hamedi and Tahami 2018) evaluated the effect of Zycosoil as an antistripping agent on the moisture susceptibility of asphalt concrete. The experimental results of surface free energy and moisture susceptibility showed that improvement in moisture resistance. (Cuadri et al. 2015) evaluated the moisture-damaged bitumen modified with thiourea and an isocyanate/castor oil prepolymer. The Rolling Bottle Test was performed to assess the moisture damage of asphalt. This showed reduced moisture susceptibility of asphalt. (Zaidi et al. 2019) conducted a comprehensive study of HL on moisture damage in bitumen mastic and asphalt mixture. The moisture damage assessment can be made by performing the Rolling Bottle Test and Bitumen Bound Strength test. Asphalt concrete modified with HL showed improvement in adhesion and resistance to moisture damage.

5. Research Methodology

In order to carry out the study of adhesion and moisture susceptibility on loose coated mixture different grades of asphalt binders and two tests, i.e. the BBS and the RBT were performed. The BBS was performed according to ASTM D 4541 and RBT was performed according to BS EN 12697-11. The commonly available grades of bitumen (60-70 and 80-100) and the modifiers wax, the HL and the LDPE were used. The aggregates from Sargodha Hills were used. The assessment and results were derived from conventional testing on, modified and controlled, binder as well as from Moisture Damage Test and Adhesion Test.
The research methodology adopted for this work is given below in Figure 1.

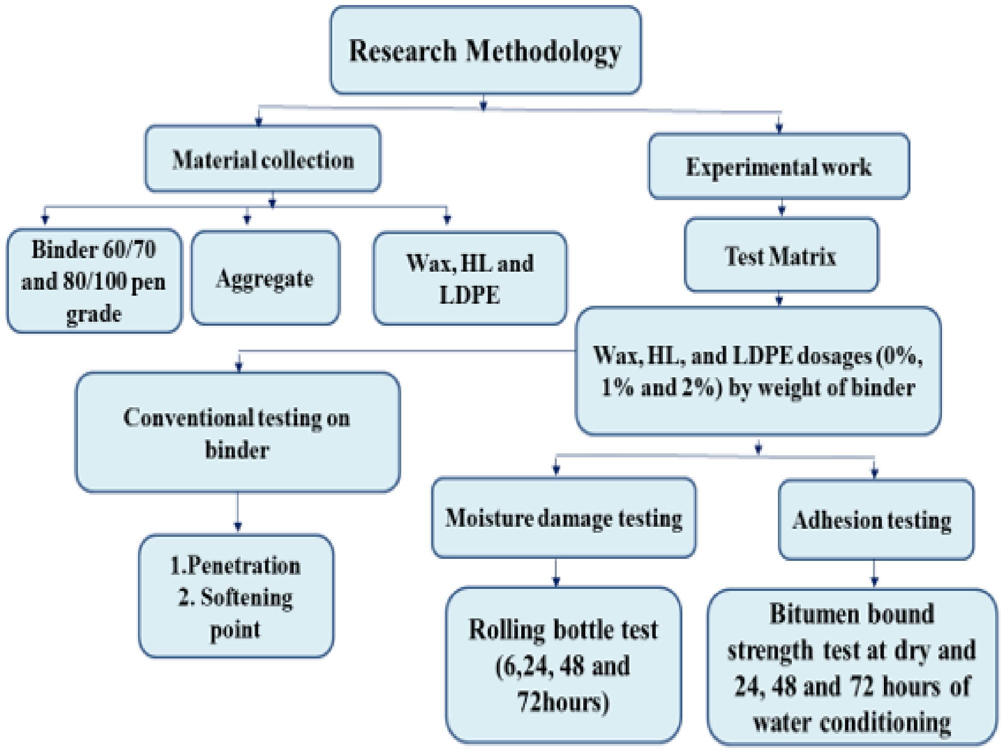

Figure 1: Research methodology

6. Results and discussion

6.1 Conventional testing

To analyze the effect of modifiers on bitumen conventional testing was performed. The purpose of performing penetration and softening point test is to check whether modified bitumen becomes soft or hard because softening and hardening of bitumen has a direct effect on adhesion and moisture damage. The experimental results of conventional testing are shown in Figures 2 and 3. Generally, with the addition of the LDPE and the HL bitumen hardens that is why the penetration value of modified bitumen decrease and softening point increases, but the addition of wax makes the bitumen soft that is why the value of penetration increases but the value of softening point decreases.

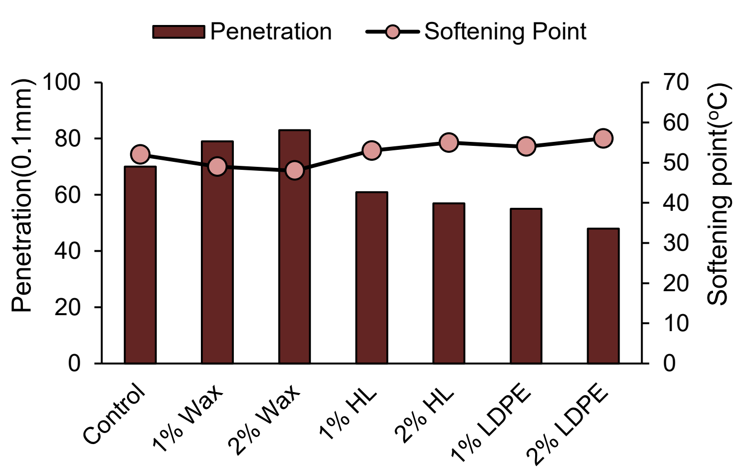

Figure 2: 60-70 pen grad modified and unmodified bitumen penetration and softening point values

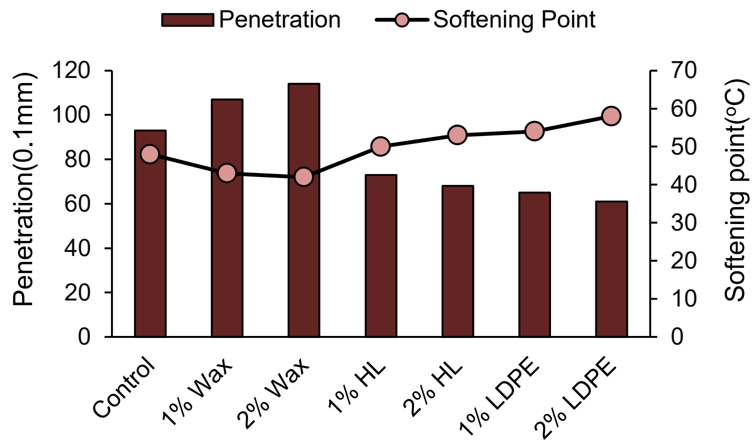

Figure 3: 80-100 pen grade modified and unmodified bitumen penetration and softening point values

The addition of 2% LDPE by weight of binder in 60-70 and 80-100 pen bitumen decreases the penetration values by 46% and 53%, whereas an increase of 8% and 21% in the softening point was observed as compared to control binder in their respective cases.

In 2% HL modified 60-70 and 80-100 pen bitumen a decrease of 23% and 36% in penetration values and an increase of 6% and 10% in values of softening point was observed as compared to control binder in their respective cases.

The addition of 2% wax by weight of binder in 60-70 and 80-100 pen bitumen penetration values increased by 18% and 22%, whereas the value of softening point decreased by 8% and 14% as compared to control binder in the respective cases.

6.2 Assessment of adhesion using BBS test

The experimental assessment of the effects of wax, HL and LDPE on adhesion, the Pneumatic Adhesion Tensile Testing Instrument (PATTI) was used. The main test advantage is that the adhesion between bitumen and aggregate can be found easily in the sense of force. All samples were tested under dry and water cured conditions (24, 48, and 72 hours). The wax, HL and LDPE with the percentages of 1% and 2% by weight binder were used in the control binder to check the bond strength of the bitumen aggregate system. The burst pressure at which stud detaches from the aggregate sample can be determined from PATTI which is then used in equation 1 to calculate Pull Off Tensile Strength (POTS).

$$\mathrm{POTS} = \frac{(\mathrm{BP} \times \mathrm{A_g}) - \mathrm{C}}{\mathrm{A_{ps}}} \qquad (1)$$

POTS is the pull-off tensile strength
BP is burst pressure
A_g is the contact area having a value of 2620 mm^2
C is the piston constant 0.286
A_{ps} is the area of pull-stub having a value of 127 mm^2, for this study F-4, stub type was used.

From Figures 2 and 3, the addition of 2% LDPE in 60-70 and 80-100 pen bitumen the POTS values increase 72% and 57% respectively at the dry condition as compared to the control binder.

The addition of 2% HL by weight of binder in 60-70 and 80-100 pen bitumen 58% and 47% POTS values increase at dry condition compare to control binder respectively.

The addition of wax in 60-70 and 80-100 pen bitumen the POTS value decreases by 6% and 2% respectively as compared to the control binder.

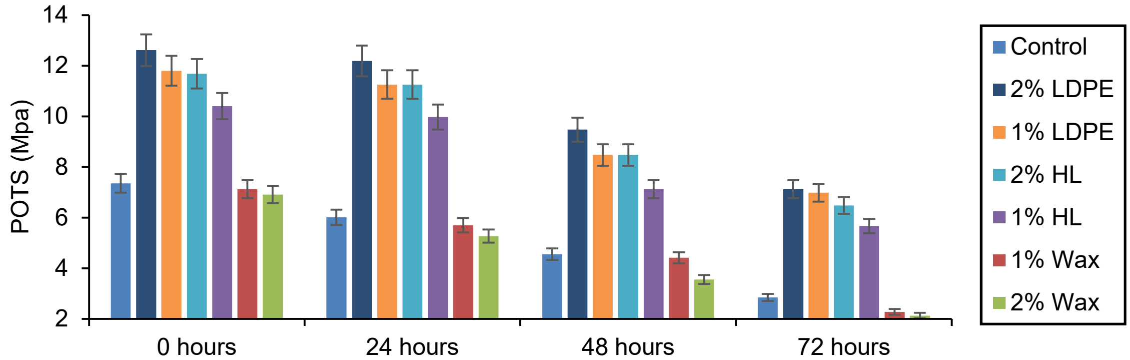

Figure 4: POTS values of 60-70 pen grade modified and unmodified bitumen at dry and wet conditioning

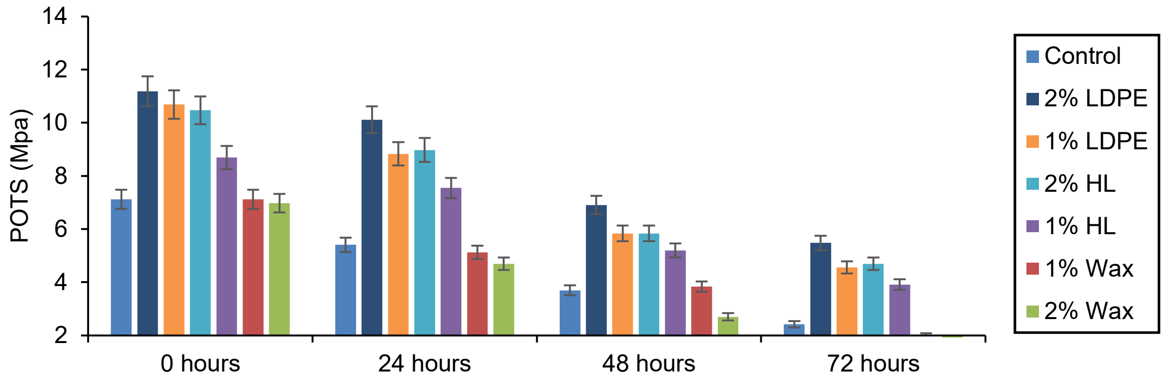

Figure 5: POTS values of 80-100 pen grade modified and unmodified bitumen at dry and wet conditioning

After 24, 48 and 72 hours of wet conditioning the POTS values of the LDPE and the HL decrease but remain higher than that of the control binder because the water penetrates the bitumen-bitumen interface and bitumen-aggregate interface which weakens the bond.

In 60-70 and 80-100 pen bitumen modified with the LDPE higher POTS values as compared to control binder were observed in dry conditions. But the LDPE modified bitumen showed lesser values after water conditioning as compared to dry conditions. HL shows lesser values of POTS in dry and water conditions as compared to LDPE but shows improved results as compared to the control binder. But in case of wax POTS values decrease as compare to control binder in both dry and wet conditions.

6.2.1 Failure surface analysis

When stub detaches from the aggregate surface, there are two types of failures, one is an adhesive failure and the other is cohesive failure. Visual identification of bitumen remains on aggregate sample determines the type of failure. When bitumen remains on the aggregate surface are greater than 50% then it is cohesive failure else it is an adhesive failure. In the case of 50% bitumen remains on aggregate the failure is cohesive-adhesive (Xie et al. 2014).

Table 1: 60/70 pen Bitumen with the percentage of coverage area and failure type

CT*	Control	1% Wax	2% Wax	1% HL	2% HL	1% LDPE	2% LDPE
0 hours	50C/A	70C	60C	60C	65C	80C	85C
24 hours	40A	55C	50C/A	55C	55C	70C	70C
48 hours	25A	35A	40A	40A	45A	55C	60C
72 hours	20A	25A	30A	25A	35A	30A	35A

CT* curing time; A, adhesive failure; C, cohesive failure; C/A, 50% adhesive 50% cohesive failure

Table 2: 80/100 pen Bitumen with the percentage of coverage area and failure type

CT*	Control	1% Wax	2% Wax	1% HL	2% HL	1% LDPE	2% LDPE
0 hours	50C/A	60C	55C	65C	70C	75C	85C
24 hours	40A	50C/A	45A	45A	60C	60C	75C
48 hours	30A	37A	35A	40A	40A	45A	50C/A
72 hours	15A	20A	25A	20A	25A	35A	40A

CT* curing time; A, adhesive failure; C, cohesive failure; C/A, 50% adhesive 50% cohesive failure

Tables 1 and 2 show the percentage of bitumen coverage and the failure type after dry and wet conditioning of the sample. In 60-70 pen bitumen modified by LDPE higher bond strengths were achieved and failure changes from cohesive to adhesive after 48 hours of water conditioning. In 80-100 pen bitumen modified by LDPE failure changes from cohesive to cohesive-adhesive after 48 hours and then to adhesive after 72 hours. Whereas in 60-70 and 80-100 pen bitumen modified by HL show higher bond strength and failure changes directly from cohesive to adhesive after 24 hours of wet conditioning.

6.3 Moisture damage assessment using RBT

The rolling bottle test was performed to measure the affinity between bitumen and aggregate.

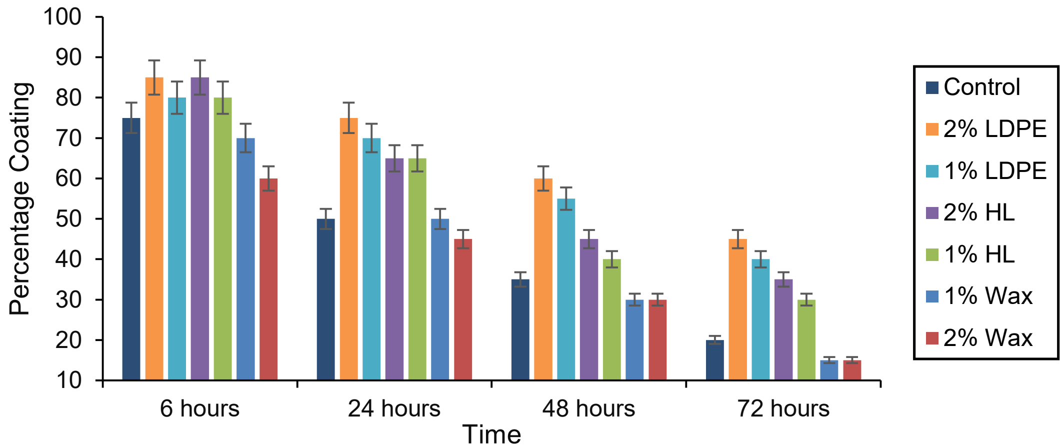

Figure 6: comparison of the percentage of bitumen coverage of 60-70 modified and unmodified bitumen at different duration

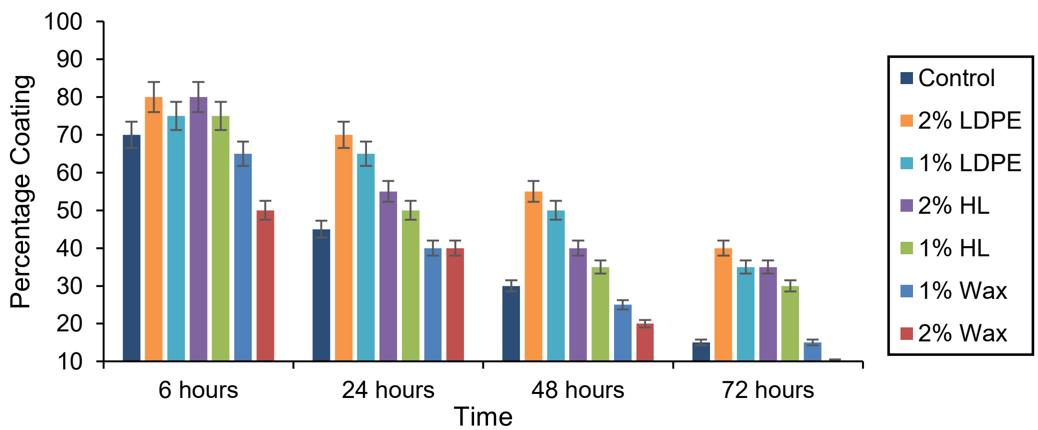

Figure 7: comparison of the percentage of bitumen coverage of 80-100 modified and unmodified bitumen at different duration

From Figures 5 and 6, it can be deduced that the increase in rolling time decreases the bitumen coverage. In the LDPE and the HL modified bitumen of 60-70 and 80-100, pen grade adhesion effect is prominent compared to the control binder. The 60-70 pen bitumen modified with LDPE and HL had 25% and 15% increased coverage as compared to the control binder after 72 hours of rolling time in their respective cases. In 80-100 pen bitumen modified with LDPE and HL had 25% and 20% more coverage as compared to control binder after 72 hours of rolling time in their respective cases. In the case of Wax, coverage reduces to 5% with respect to controlled binder leading to increased stripping.

7. Relationship between BBS and RBT test results

Figure 8. shows a linear correlation between bitumen coverage of BBS and RBT tests results. R^2 values vary from 0.814 to 0.842 in BBS and RBT test results. Higher values of the coefficient of determination suggest that bitumen coverage values of two tests are strongly correlated with each other. Moreover, the P-value is much smaller from the level of significance which shows that a statistically significant correlation exists.

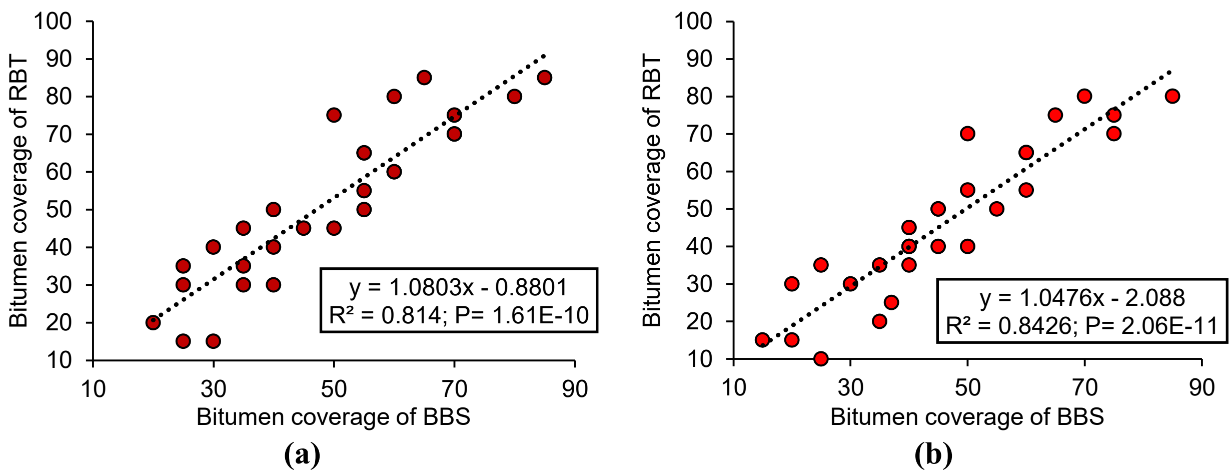

Figure 8: Correlation between bitumen coverage of BBS and RBT tests results of (a) 60-70 pen bitumen and (b) 80-100 pen bitumen

8. Conclusion

The conventional, BBS and RBT tests were performed in the study to examine the adhesion and moisture damage resistance. The 60-70 and 80-100 pen bitumen modified with wax, the HL and the LDPE have been investigated. The following main conclusions were drawn from this study;

- With the addition of 2% LDPE and the HL in 60-70 and 80-100 pen bitumen penetration values decrease by 23%-53% whereas the values of softening point increase by 6%-21% respectively as compared to control binder.
- In 60-70 and 80-100 pen bitumen, with the addition of 2% LDPE, the POTS values increase by 72% and 57% respectively whereas with the addition of the HL, the POTS values increase by 58% and 47%. On the other hand, with the addition of 2% wax the POTS values decrease by 6% and 2% respectively at the dry condition.
- In 60-70 pen bitumen modified by LDPE higher bond strengths were achieved and failure changes from cohesive to adhesive after 48 hours of water conditioning. In 80-100 pen bitumen modified by LDPE failure changes from cohesive to cohesive-adhesive after 48 hours and then to adhesive after 72 hours. Whereas in 60-70 and 80-100 pen bitumen modified by HL show higher

bond strength and failure changes directly from cohesive to adhesive after 24 hours of wet conditioning.

- In LDPE and HL modified bitumen of 60-70 and 80-100 pen grade percentage of bitumen coverage increases by 15%-25% and 20%-25%, respectively, as compared to control binder after 72 hours of rolling time. In the case of Wax, bitumen coverage reduces to 5% as compared to a controlled binder which leads to increased stripping.

9. Future recommendations

The type of failure in the BBS test and bitumen coverage in RBT has visually estimated thus chances of error are there. By using the software, the bitumen coverage and the failure type can be determined more accurately.

References

Ali, Hend, Nur Izzi, Halil Ceylan, Zainuddin Sajuri, and Fauzan Mohd. 2016. "Investigation of the Relationship between Fluidity and Adhesion Strength of Unmodified and Modified Bitumens Using the Pull-off Test Method." *Construction and Building Materials* 122: 140–48. https://doi.org/10.1016/j.conbuildmat.2016.06.053.

Cihlářová, Denisa, Ivan Fencl, Silvia Cápayová, and Petr Pospíšil. 2018. "Use of Adhesion Promoters in Asphalt Mixtures." *Slovak Journal of Civil Engineering* 26 (1): 19–24. https://doi.org/10.2478/sjce-2018-0003.

Cuadri, A. A., P. Partal, N. Ahmad, J. Grenfell, and G. Airey. 2015. "Chemically Modified Bitumens with Enhanced Rheology and Adhesion Properties to Siliceous Aggregates." *Construction and Building Materials* 93: 766–74. https://doi.org/10.1016/j.conbuildmat.2015.05.098.

Hamedi, Gh H., and S. A. Tahami. 2018. "The Effect of Using Anti-Stripping Additives on Moisture Damage of Hot Mix Asphalt." *International Journal of Adhesion and Adhesives* 81 (January 2018): 90–97. https://doi.org/10.1016/j.ijadhadh.2017.03.016.

Liu, Yawen, Alex Apeagyei, Naveed Ahmad, James Grenfell, and Gordon Airey. 2014. "Examination of Moisture Sensitivity of Aggregate-Bitumen Bonding Strength Using Loose Asphalt Mixture and Physico-Chemical Surface Energy Property Tests." *International Journal of Pavement Engineering*. Taylor & Francis. https://doi.org/10.1080/10298436.2013.855312.

Solaimanian, Mansour, John Harvey, Maghsoud Tahmoressi, and Vivek Tandon. 2003. "Test Methods to Predict Moisture Sensitivity of Hot-Mix Asphalt Pavements." In *Moisture Sensitivity of Asphalt Pavements-A National SeminarCalifornia Department of Transportation; Federal Highway Administration; National Asphalt Pavement Association; California Asphalt Pavement Alliance; and Transportation Research Board.*

Xie, Jun, Zongwu Chen, Ling Pang, and Shaopeng Wu. 2014. "Implementation of Modified Pull-off Test by UTM to Investigate Bonding Characteristics of Bitumen and Basic Oxygen Furnace Slag (BOF)." *Construction and Building Materials* 57: 61–68. https://doi.org/10.1016/j.conbuildmat.2014.01.083.

Zaidi, Syed Bilal Ahmed, Gordon D Airey, James Grenfell, Rami M Alfaqawi, Imtiaz Ahmed, Naveed Ahmad, and Mike Haynes. 2019. "Moisture Susceptibility of Hydrated Lime Modified Mastics Using Adhesion Test Methods and Surface Free Energy Techniques." *International Journal of Pavement Engineering*, 1–13.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Use of admixtures in improving bonded asphalt overlay on concrete pavements-A critical review

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Abstract
The bonding between concrete pavements and asphalt overlay is considered a critical factor in deciding the performance of both concrete and asphalt overlay. It is observed that problems with the interface bonding cause different issues like slippage cracking and rutting due to lateral movement of overlay asphalt. These distresses further affect the bridge deck by allowing water infiltration and impact loading on uneven surface on bridge decks. In this regard**,** admixtures are being assessed to improve the bonding performance of tack coat. Given this background, the aim of this review paper is to document the use of emerging admixtures in tack coat layer and asphalt overlay that would be largely implemented to increase bonding between concrete and asphalt layer. This review identified the different admixtures which can improve the interface bonding and adhesion also providing good water resistance. In addition, different kind of asphalt modifiers, asphalt binders and different tests to assess the effectiveness of these modifiers and admixtures were also reviewed critically. This review concluded with few recommendations in terms of effective admixtures to be used on the basis of good performance, economy, ease of application and environment friendliness.

Keywords
Bonded asphalt overlay; Tack coat; Trackless tack coat; Asphalt modifiers; Admixtures.

1. Introduction

The use of asphalt overlay over concrete pavements is being used from a long time and has been proved to be very efficient method to provide skid resistance, water proofing to concrete pavement below and saving it from deterioration. Concrete pavements with asphalt overlays are prone to different defects and cracking and it is important to indicate these distresses and their causes but most importantly there is a need to develop strategies to avoid these defects in new pavements which are to be constructed. It is observed that most of these issues arising in asphalt overlays are due to the weak bonding between the concrete pavement and the asphalt overlay. The provision of an effective bonding layer to increase the bonding between the two materials makes both layers to act as a single unit hence increasing the strength of the pavement furthermore the asphalt layer also helps in distribution of vehicle load through a larger area by the overlay & therefore decreasing the wheel load stress. Moreover, the tack coat layer used between the concrete pavements or concrete bridge decks and asphalt overlay also acts as a water proofing layer. There are a lot of issues that arise on the bridge decks which include slippage cracking and other issues related to the bondage between the concrete deck and asphalt overlay. The substantial influence of interlayer bonding between pavement layers on the response of pavements in terms of stresses due to traffic loading have been illustrated by many researchers. The traffic stress will disperse from one layer of the pavement into the -layer below it if the bonding between the layers is sufficient, in case of insufficient bonding between the layers and the application of different unfavorable loads increases the stresses at the bottom of layers these stresses are tensile in nature which may result in different kind of distresses such as slippage cracking, pot holes, cracking and raveling.

In the course of time different materials have been used as tack coat for increasing the bondage between the asphalt overlay and concrete pavements. Asphalt emulsion is considered to be the most widely used tack coat across the globe according to a worldwide survey (Mohammad et al., 2008). However conventionally used waterproofing adhesive layers have issues that are related to temperature. As the temperature increases the effectiveness of these tack coats reduce. Different types of Tack coats used to increase bondage between bridge deck and asphalt overlays becomes dysfunctional with increasing temperature (Hag-Elsafi O. et al., 2001). The use of these conventional tack coat reduces the performance of overlays causing premature failure ultimately causing wastage of material hence emerging technology to reduce this wastage and to increase the performance is required. This review research carried out on different modifiers and admixtures helps in classifying them according to their vulnerability to temperature effect, performance as tack coat layer, increase in stiffness and flexural strength and their adverse effect on environment e.g. asphalt types like rapid curing and medium curing cutback asphalt which are highly volatile and cause environmental issue irrespective of the fact that they have good performance.

2. Flaws in bonded asphalt overlay on concrete pavements

The flaws and distresses on a pavement results in development of unsmooth surface of the road which not only cause discomfort to the users but also increases the risk of accidents of vehicles (Granlund, 2012). The safety and performance of concrete pavements on bridges is not only decided by the design and construction of the structural member but also on the basis of performance of the asphalt overlay added to protect the concrete layer below, to provide a smooth riding quality by providing a plane finish and protect against water penetration and other materials that can harm the pavement is also a factor that decides the safety and performance (El Halim et al., 2017). The most common types of problems or distresses on concrete pavements with asphalt overlays observed are cracking, scaling, spalling and reinforcement corrosion. Mostly the corrosion of reinforcement is assocciated with the water seepage from asphalt overlay into the concrete pavement, moreover delamination occurs due to the fact that two different layers of concrete and asphalt exist in a concrete pavement with asphalt overlay (Yehia et al., 2007).The defects in the concrete layer or pavement arise when the asphalt overlay fails to serve its purpose when the asphalt overlay experiences defects.

If the interfacial bonding between the concrete pavement and asphalt overlay is not sufficient and unfavourable load conditions act on the pavement then the tensile stresses and strains increase at the bottom of the respective layers, this results in distresses which are premature. These distresses include slippage cracking, potholes, deformation, cracking, bulging and ravelling (Wang et al., 2017). The premature failure including cracking, potholes, swelling and slippage cracking is due to the shear failure of the waterproofing adhesive layer. This shear failure of WAL can happen due to difference of thermal coefficient, which leads to differential expansion hence causing failure of WAL (Li et al., 2008). The working cracks in the existing pavements proliferate upwards from older overlay to new overlay and becomes the cause of reflective cracking which is the most critical reason of deterioration. One of the major distress that affects the pavement overlay is reflective cracking. Cracks from the bottom layer propogate towards the overlay when the bottom layer induces stress concentration adjacent to the crack tip (Baek and Al-Qadi., 2006). High temperature service condtions related with rutting distress is a key issue which affects the the pavemment in terms of long term servicibility which gives rise to ride quality problems (Kim et al., 2018) Fatigue cracking and rutting damge are key type of distresses which requires critical attention as these types of damages affect the serviceability as well as the safety over the performance life of asphalt layer. Rutting is dominant at high temperatures whereas fatigue cracking occurs in more low and medium temperature range (Bazzaz et al., 2018). The four main type of distresses which are very common and are used to assess the performance of any test section are fatigue cracking, longitudinal cracking in the wheel paths, rutting and roughness (Mousa et al., 2019).

Table 1: Flaws in bonded asphalt overlay on concrete pavements

Flaws	Literature
Slippage cracking	(Wang et al., 2017), (Li et al., 2008)
Potholes	(Wang et al., 2017), (Li et al., 2008)
Bulging	(Wang et al., 2017)
Raveling	(Wang et al., 2017)
Deformation	(Wang et al., 2017)
Rutting	(Kim et al., 2018), (Bazzaz et al., 2018), (Mousa et al., 2019)
Reflective cracking	(Baek and Al-Qadi**.**, 2006)
Fatigue cracking	(Yehia et al., 2007), (Li et al., 2008), (Bazzaz et al., 2018), (Mousa et al., 2019)

3. Available admixtures for improving the bond between asphalt overlay and concrete pavements

Admixtures are materials added to incorporate some desired properties that a material lacks. The purpose of adding admixture can be to increase the tensile strength of the asphalt overlay, increase the stiffness of the asphalt or to increase the bonding performace of the asphalt tack coat layer.

3.1 Modified tack coat

ASTM D8-02 defines tack coat as the application of an asphalt emulsion or other liquid asphalt that is placed on an existing asphalt overlay or portland concrete cement layer. Commonly HMA cut back asphalt and asphalt emulsion are used as tack coat materials hoever cut back asphalt is avoided due to the environmental issues related to it. (Mohammad et al., 2012). Out of theses tack coat types the most commonly used is asphalt emulsion and it has the ability to be improved by addition of modifiers. (Canestrari et al 2005). By adding styrene-butadiene-styrene, Mastic uintaite and anti rut asphalt a modified asphalt trackless tack coat material was prepared it was found that the performance of bonding of this modified tack coat was better than conventional tack coats i.e hot mix asphalt tack coat. (Hou et al., 2018). The addition of mixtures like sytyrene and butadiene not only increase the shear strength of the tack coat but the water resisting quality and the capability to resist extreme high and low temperatures also increases (Li et al., 2019).

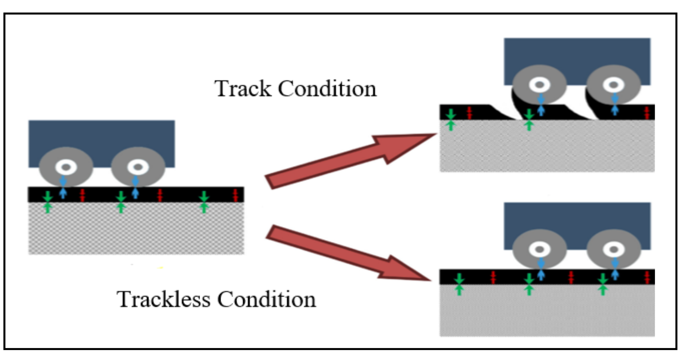

Figure 1: Tyre tracking phenomenon (Hou et al., 2018)

3.2 Fibre reinforced asphalt concrete

One approach taken to improve the pavement performance is the modification of the asphalt binder. The use of fibres to improve the performance of pavements is an effective technique to stop running off aggregate during construction because it bulks the asphalt, moreover it also works as a filler material, the optimum binder content also shows a slight increment with the addition of fibres (Abtahi et al, 2010). Polymers and fibres are two materials used in FRAC, however according to different researchers fibres are considered more effective among various modifiers.

3.2.1 Polypropylene fibres

The comparison of polypropylene fibres added in asphalt overlays with that of normal asphalt overlays after testing concluded that fibre modified samples were stiffer and their fatigue life was also more as compared to asphalt mixture without any modifier. The major issue that came across was that the polypropylene fibers characteristics were conflicting with hot asphalt binder due to the fact that these fibres have low melting point (Huang and White, 1996).

3.2.2 Carbon fibre

For the modification of asphalt binder, it is believed that carbon fibres are more advantageous than other types of fibres this is due to the reason that these fibres are composed of carbon and asphalt is a hydrocarbon which means that they are compatible characteristically. The melting of the fibre is not an issue in this case as carbon fibers are manufactured at enormously high temperatures i.e. over 1000 C. The main advantage of carbon fibre used in asphalt overlays is that it has very high tensile strength which is why it also increases the tensile strength of asphalt overlay and related properties like resistance to thermal cracking also increases of AC mixtures, including resistance to thermal cracking (Chung et al. 2012).

3.3 Asphalt modifiers for wet-freeze climate

Asphalt modifiers for the purpose of increasing the performance of the asphalt in wet-cold regions is an important aspect as the performance of asphalt overlays decreases as the temperature increases or decreases below some limit. Cracking occurs as the temperature reduces hence it is required to use such admixtures or modifiers that would increase the performance of the asphalt overlay in cold regions.

3.3.1 Anti-stripping agents

The use anti-stripping agents is currently in practice for wet-freeze climate regions. This kind of modifier helps to bind the asphalt with the aggregate in a better way and prevents stripping of binder from the aggregate, anti-stripping agent is mainly used for HMA, but it can also help with asphalt pavements constructions using warm-mix (Hicks et al., 2003).

3.3.2 Rubber-modified binder

The recycled tires can be used to extract crumb rubber and this crumb rubber can be used to modify the asphalt called the crumb rubber modified asphalt. (Ge et al., 2016). The composition of crumb rubber usually consists of three different materials these are natural rubber, carbon black and synthetic rubber. Here the carbon black is byproduct which is obtained by the incomplete combustion of petroleum products. The use of crumb rubber is not only restricted to surface layer but also can be used in structural layer asphalt (Huang et al., 2002).

3.3.3 Polyphosphoric acid

Polyphosphoric acid (PPA) is known to improve the performance, at high temperature, of the pavement against rutting distress and also increases the performance at very low temperature to mitigate cracks developed due to cold climate of the region. PPA when used to modify the asphalt binder chemically can improve the low temperature as well as high temperature without leading to oxidation of the asphalt. (Ge et al., 2017). A survey was conducted which revealed that PPA can be used in a quantity of 0.5% for high volume roads and for low volume roads 1% of PPA can be used (Fee et al., 2010).

4. Performance of asphalt overlay on concrete pavements with admixtures

The use of CRMA in different cold regions was surveyed for the performance in USA, China and Canada, it was seen in the survey that the use of crumb rubber asphalt provided longevity of open graded friction course moreover there was reduction in wet weather accidents and noise when used with OGFC (Cheng et al., 2012). The performance of the SBR modified latex asphalt for use in tack coat was evaluated by using a 45-degree oblique shearing test of core samples and was found that the addition of SBR latex and epoxy improves the shear strength of the tack coat significantly moreover it was also found that addition of such modifier also help cope with the extreme high and low temperatures. However, the results were achieved with optimum SBR and epoxy content (Zhang et al., 2017). The performance of fibre reinforced asphaltic concrete was also evaluated the type of fibre used were of many types however, the use of steel fibres with waste tyre rubber for environmental sustainability and increasing performance resulted in mitigation of loss in flexure strength due to the content of rubber in it. The use of rubberized concrete also enhanced the strain capacity and also the post-peak strain capacity (Alsaif et al., 2018). The performance of SBS modified asphalt was evaluated using an influential indicator for high temperature performance. The evaluation was done using the creep recovery test index, dynamic viscosity and softening point. It was found that the SBS modifier content is the most critical factor. The addition of SBS modifier increases the high temperature resistance (Yang et al., 2019). The performance of PPA and Sasobit after evaluation using different testing proved to increase the rheological characteristics, the penetration of asphalt decreased, and softening point increased which proves that the deformation resistance was improved (Gee et al., 2017).

5. Discussion

Over the past few decades, different admixtures have been used for the improvement in the performance of the concrete pavements having bonded asphalt overlay. Based on the reviewed literature different results are presumed and recommendations are given, for example dissimilar climatic regions the tack coat will behave in a different manner i.e. in high temperature regions the thermal expansion will occur in a greater rate as compared to regions of low temperature, this variable expansion of concrete, tack coat and asphalt overlay will result in debonding of layers. Asphalt overlays are vulnerable in wet and cold regions and to decrease this susceptibility chemical addition of Polyphosphoric acid in the asphalt overlay should be opted for cold and wet regions reason being that polyphosphoric acid is a good dehydrating agent. The use of modern modifiers like SBS tack coat and NTCRS-1hM should be opted for hot climate regions the reason being that they help in reducing tracking condition of the asphalt overlays. Cracking and rutting being the most common type of distresses on bonded asphalt overlays which can be avoided using fibres in the asphalt overlay. The use of carbon fibres is recommended due to their dominance over other types of fibres on the basis of strength and melting point. Sasobit should be used as a modifier as it allows early traffic release hence saving the curing time. In addition to this Sasobit also increases the softening point of the bitumen and the stability of asphalt.

6. Conclusion

The bonded asphalt overlay on concrete increases the serviceable life and ride quality of the pavement and If the overlay gets damaged, the bottom concrete pavement is affected. It is important to find solutions for the bonded asphalt overlay to increase its performance. Different admixtures can be added to the overlay to increase different characteristics which include resistance to extreme high and low temperatures, resistance to flexure stresses, increase in bonding between concrete and asphalt layer. Every admixture has its own characteristic, and at the same time it, also has issues related to it. For the purpose of increasing the bonding between the layers, SBS modified asphalt is considered comparatively better option as it does not have any environmental issues related to it. Similarly, for temperature resistance, crumb rubber asphalt is a good option as it uses recycled rubber and the temperature tolerance is greater; moreover, it can also be used on structural layer. For the purpose of increasing the stiffness of the asphalt overlay carbon fibre is a fine option; however, the use of recycled steel fibres is even better option keeping in mind the strength it provides and the reason that it is a more sustainable option.

7. Acknowledgments

The authors would like to acknowledge the efforts and help of Muhammad Ali Tariq and Syed Ali Zulnorain. The careful review and constructive suggestion by the anonymous reviewer is gratefully acknowledged.

8. References

Abtahi, S. M., Sheikhzadeh, M., & Hejazi, S. M. (2010). Fiber-reinforced asphalt-concrete–a review. *Construction and Building Materials*, *24*(6), 871-877.

Alsaif, A., Koutas, L., Bernal, S. A., Guadagnini, M., & Pilakoutas, K. (2018). Mechanical performance of steel fibre reinforced rubberised concrete for flexible concrete pavements. *Construction and Building Materials*, *172*, 533-543.

Baek, J., & Al-Qadi, I. L. (2006). Finite element method modeling of reflective cracking initiation and propagation: Investigation of the effect of steel reinforcement interlayer on retarding reflective cracking in hot-mix asphalt overlay. *Transportation Research Record*, *1949*(1), 32-42.

Bazzaz, M., Darabi, M. K., Little, D. N., & Garg, N. (2018). A straightforward procedure to characterize nonlinear viscoelastic response of asphalt concrete at high temperatures. *Transportation Research Record*, *2672*(28), 481-492.

Canestrari, F., & Santagata, E. (2005). Temperature effects on the shear behaviour of tack coat emulsions used in flexible pavements. *International Journal of Pavement Engineering*, *6*(1), 39-46.

Cheng, D., & Hicks, R. G. (2012). Survey on the Use of Rubber Modified Asphalt Pavements in Cold Regions.

Chung, D. D., & Chung, D. (2012). *Carbon fiber composites*. Elsevier

El Halim, A. A., El-Desouky, A., & El Halim, A. (2017, July). Extending the Service Life of Bridges Through Proper Compaction of Asphalt Decks. In *International Congress and Exhibition" Sustainable Civil Infrastructures: Innovative Infrastructure Geotechnology"* (pp. 49-65). Springer, Cham.

Fee, D., Maldonado, R., Reinke, G., & Romagosa, H. (2010). Polyphosphoric acid modification of asphalt. *Transportation Research Record*, *2179*(1), 49-57.

Ge, D., Yan, K., You, L., & Wang, Z. (2017). Modification mechanism of asphalt modified with Sasobit and Polyphosphoric acid (PPA). *Construction and Building Materials*, *143*, 419-428.

Ge, D., Yan, K., You, Z., & Xu, H. (2016). Modification mechanism of asphalt binder with waste tire rubber
and recycled polyethylene. *Construction and Building Materials*, *126*, 66-76.

Hicks Gary, R., Santucci, L., & Ashenbrener, T. (2003). Introduction and Seminar Objectives. Moisture Sensitivity of Asphalt Pavements: A National Seminar. *Transportation Research Board of the National Academics, San Diego, California*, 3-20.

Hou, D., Han, M., Muhammad, Y., Liu, Y., Zhang, F., Yin, Y., ... & Li, J. (2018). Performance evaluation of modified asphalt based trackless tack coat materials. *Construction and Building Materials*, *165*, 385-394.

Huang, B., Mohammad, L. N., Graves, P. S., & Abadie, C. (2002). Louisiana experience with crumb rubber-modified hot-mix asphalt pavement. *Transportation Research Record*, *1789*(1), 1-13.

Huang, H., & White, T. D. (1996). Dynamic properties of fiber-modified overlay mixture. *Transportation Research Record*, *1545*(1), 98-104.

Kim, S. M., Darabi, M. K., Little, D. N., & Al-Rub, R. K. A. (2018). Effect of the realistic tire contact pressure on the rutting performance of asphaltic concrete pavements. *KSCE Journal of Civil Engineering*, *22*(6), 2138-2146.

Li, J., Liu, J., Zhang, W., Liu, G., & Dai, L. (2019). Investigation of thermal asphalt mastic and mixture to repair potholes. *Construction and Building Materials*, *201*, 286-294

Li, Y., Li, S., Lv, R., Zhang, P., Xu, Y., Hou, G., & Cui, C. (2015). Research on failure mode and mechanism of different types of waterproof adhesive materials for bridge deck. *International Journal of Pavement Engineering*, *16*(7), 602-608.

Mohammad, L. N. (2012). *Optimization of tack coat for HMA placement* (Vol. 712). Transportation Research Board.

Mousa, M., Elseifi, M. A., & Abdel-Khalek, A. (2019). Development of Tree-Based Algorithm for Prediction of Field Performance of Asphalt Concrete Overlays. *Journal of Transportation Engineering, Part B: Pavements*, *145*(2), 04019011.

Yang, Y., Shao, H. J., Zhou, W., & Jin, G. L. (2019). Evaluation on the High Temperature Performance Indices of SBS Modified Asphalt. In *CICTP 2019* (pp. 1011-1022).

Yehia, S., Abudayyeh, O., Nabulsi, S., & Abdelqader, I. (2007). Detection of common defects in concrete bridge decks using nondestructive evaluation techniques. *Journal of Bridge Engineering*, *12*(2), 215-225.

Wang, J., Xiao, F., Chen, Z., Li, X., & Amirkhanian, S. (2017). Application of tack coat in pavement engineering. *Construction and Building Materials*, *152*, 856-871.

Zhang, Q., Xu, Y. H., & Wen, Z. G. (2017). Influence of water-borne epoxy resin content on performance of waterborne epoxy resin compound SBR modified emulsified asphalt for tack coat. *Construction and Building Materials*, *153*, 774-782.

11th International Civil Engineering Conference (ICEC-2020) "Integrating Innovation & Sustainability in Civil Engineering" March 13-14, 2020, Karachi, Pakistan.

Assessment of asphalt overlay performance over concrete bridge deck

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Abstract

The use of asphalt overlay for rehabilitation of pavements has been practiced from early nineties; however, they are also used on concrete bridge decks to protect the underlying concrete bridge deck and increase the riding quality to ensure comfort for the road user. The asphalt overlay is vulnerable to different kinds of defects and flaws. These flaws not only compromise the road user's comfort, but also causes damage to the concrete bridge deck below. The aim of this paper is to assess the performance of asphalt overlay and to report different distresses that arise on it. This assessment will not only help in maintenance planning of these bridges, but also help in finding possible solutions to these distresses. A thorough field work is conducted using the Federal Highway Authority pavement condition index Performa. This Performa indicates the distresses as well as evaluates the condition of the asphalt overlay by assessing the extent and severity of distresses. Concrete bridges with asphalt overlay having different level of damages are inspected. This study concludes with indication of most occurring distresses on asphalt overlays over concrete bridge decks and recommendations for the purpose of efficient remedial measures for these distresses.

Key Words

Rutting, Bleeding, Settlement, longitudinal and transverse cracking, Asphalt overlay

1.Introduction

Different materials have been used as tack coat for increasing the bondage between the asphalt overlay and concrete pavements. Asphalt emulsion is considered to be the most widely used tack coat across the globe according to a worldwide survey (Mohammad et al.2008). However conventionally used waterproofing adhesive layers having temperature related issues. As the temperature increases the effectiveness of these tack coats reduce. Different types of tack coats used to increase bondage between bridge deck and asphalt overlays becomes dysfunctional with increasing temperature (Hag-Elsafi et al. 2001). The use of these conventional tack coat reduces the performance of overlays causing premature failure ultimately causing wastage of material hence emerging technology to reduce this wastage and to increase the performance is required. The current research is being carried out on different modifiers and admixtures which are less vulnerable to temperature effect but are also environment friendly unlike some asphalt typeslike rapid curing and medium curing cutback asphalt which highly volatile and cause environmental issue irrespective of the fact that they have good performance.

Bonded Asphalt overlay is hot mix asphalt (HMA) layer used mainly for the rehabilitation of damaged flexible pavements however they are also used in concrete and steel bridge decks. In Concrete bridge decks the asphalt overlay is used for different purposes these include smooth riding quality, water proofing of the bridge deck below, skid resistance to the vehicles. When bituminous overlay is provided over a concrete bridge deck, the wheel load is distributed through a larger area by the overlay & therefore wheel load stress is slightly decreased on bridge deck. Furthermore, the maximum temperature differential in the concrete deck is also decreased due to bituminous overlay, thus causing a substantial reduction in the warping stress thus there is a drop in the maximum value of combined stress due to wheel load and warping. The thermal coefficient of concrete and asphalt are different which also leads to differential to thermal expansion. This type of expansion causes bondage problems and also is a reason for cracking. The shear strength and the shear reaction modulus increase with decreasing temperature and increasing normal stress levels (Yao et al. 2016). Asphalt Overlays are used on concrete bridge decks to increase the serviceability and permeability.

Interface bonding between the reinforced cement concrete (RCC) bridge deck and asphalt overlay is a substantial and important point affecting mechanical behavior and service life of the pavement. Interface bonding conditions between pavement layers have a major impact on pavement performance (Mousa et al. 2017). Many factors may influence the interfacial adhesive capacity, including asphalt mixture type and tack coat type, surface texture on layer below, temperature and moisture conditions. It is observed that these problems with the interface bondage causes different issues like slippage cracking and rutting due to lateral movement. These cracks further affect the bridge deck by allowing water seepage and impact loading on uneven surface on bridge decks hence it is important to access the bonding issues and then develop strategies to solve the issue. In this project we will examine the distresses that are inspected during the survey and the best possible rehabilitations measures further propose the economical construction practices for future development.

2. Methdology

2.1. Performa

The rating methodology relies upon visual review of pavement distress. though the link between pavement distress and performance is not well outlined, there is general agreement that the flexibility of a pavement to sustain traffic heavy traffic loads during a safe and sleek manner is adversely stricken by the prevalence of noticeable distress. The rating methodology provides a procedure for uniformly characteristic and describing, in terms of severity and extent, pavement distress. The mathematical expression for pavement condition rating (PCR) provides associate degree index reflective the composite effects of variable distress varieties, severity, and extent upon the condition of the pavement. The model for computing PCR relies upon the summation of deducts points for every form of noticeable distress. Deduct values area unit a perform of distress sort, severity, and extent. Deduction for every distress sort is calculated by multiplying distress weight times the weights for severity and extent of the distress. Distress weight is that the most range of deductible points for every completely different distress sort. The distress which were visualized in on the bridge deck slab were present in the Performa and the Performa is also telling us the extent of the distress that what is extent either low, medium, high or extreme. To repair and eliminate the defects it is important to first identify different types of defects that occur on concrete bridge decks with asphalt overlays. Different type of defects are seen on different types of bridges. These defects range from mild overlay cracking to extreme overlay deterioration. To prepare a prototype in laboratory and to decide the type of testing it is important to first assess the types of distresses and their intensity. It is quite important to find out the probable cause of the pavement failure being investigated. The probable causes are normally stated, and there are often multiple factors that contributed to the failure. The first stage in determining the failure cause is the investigative synthesis, where all the information gathered is listed. From this listed information, it is then necessary to determine which information supports or refutes each of the possible failure hypotheses. This may be initially done by considering general failure causes, such as those related to construction, materials, design, or the environment. It is more required that specific cause of the failure be considered. This is achieved by going through the possible failure cause for the distress type. These all flaws were analyzed by using FHWA Performa and then PCR will be calculated by using the standard procedure as by FWHA, figure-1is taken from the FHWA manual for the inspection of the bridges.

Section: ______________ Date: ________________

Log mile: _____ to_______ Rated by: ______________

Sta: ________ to_______ ________________

PAVEMENT CONDITION RATING FORM

DISTRESS	DISTRESS WEIGHT	SEVERITY WT.*			EXTENT WT.**			DEDUCT POINTS***
		L	M	H	O	F	E	
RAVELING	10	0.3	0.6	1	0.5	0.8	1	
BLEEDING	5	0.8	0.8	1	0.6	0.9	1	
PATCHING	5	0.3	0.6	1	0.6	0.8	1	
POTHOLES/DEBONDING	10	0.4	0.7	1	0.5	0.8	1✓	
CRACK SEALING DEFICIENCY	5	1	1	1	0.5	0.8	1	
RUTTING	10	0.3	0.7	1	0.6	0.8	1✓	
SETTLEMENT	10	0.5	0.7	1	0.5	0.8	1	
CORRUGATIONS	5	0.4	0.8	1	0.5	0.8	1	
WHEEL TRACK CRACKING	15	0.4	0.7	1	0.5	0.7	1✓	
BLOCK AND TRANSVERSE CRACKING	10	0.4	0.7	1	0.5	0.7	1✓	
LONGITUDINAL JOINT CRACKING	5	0.4	0.7	1	0.5	0.7	1	
EDGE CRACKING	5	0.4	0.7	1	0.5	0.7	1	
RANDOM CRACKING	5	0.4	0.7	1	0.5	0.7	1✓	

*L = LOW **O = OCCASIONAL TOTAL DEDUCT =

M = MEDIUM F = FREQUENT SUM OF STRUCTURAL DEDUCT (✓) =

H = HIGH E = EXTENSIVE **100 - TOTAL DEDUCT = PCR =**

Figure-1: Assessment Performa for bridge deck (FHWA manual)

2.2. Selection of sites

The site selection was done randomly to observe the failures or flaws in bridge deck. This initially survey of five bridges was conducted to check that which types of flaws are most common in the bridge deck. Planning is vital to make sure that scrutiny and analysis of pavement failures were dispensed their supposed ta sks at intervals an inexpensive timeframe and at all-time low price. once coming up with the analysis program, a general review of the matter ought to initial be conducted, beside the attainable scope of scrutiny and maintenance work which will have to be compelled to be dispensed. This set up ought to be written, addressing goals, budgeting constraints, operations coming up with and therefore the inquiring synthesis. The technical team ought to be determined upon. A Survey Performa will be made which will be used to identify the different defects on the bridge decks with asphalt overlays. The Performa will contain the possible defects which occur in the bridge decks. The Performa will have a separate column for the severity of the defect. This Performa will help in recording the defects that were observed on concrete bridge decks with asphalt overlays. Almost 15 concrete bridges with asphalt overlays will be surveyed in the Rawalpindi and Islamabad region, and defects will be recorded using visual observation.

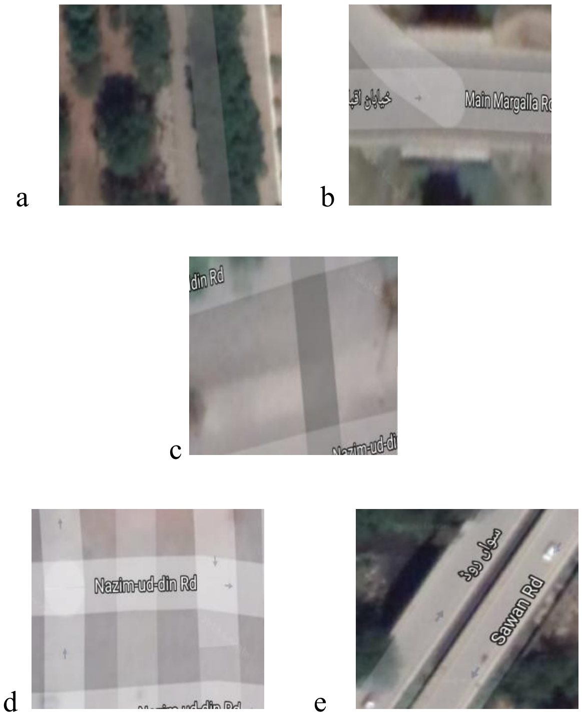

Figure-2: selection of bridges sites

a) Beam bridge b) Beam bridge c) Beam bridge d) Beam bridge e) Beam bridge

2.3. Analysis to be preformed

The pavement condition survey might embody visual examination of pavement failures, the effectiveness of evacuation structures and alternative details like topography and alignment ought to be recorded, and also the soil and earth science of the encompassing areas can also be of importance in crucial the causes of the pavement failure. an efficient visual survey of pavement failures is crucial, to confirm that the reason for the failure will be diagnosed with efficiency and it's a guide to what testing ought to be administrated and wherever. additionally, it'll give valuable web site data which will have an influence on the most effective maintenance operation. Distresses measurement ought to be administrated on failing pavement sections to seek out the quantity, type, and condition or severity level of distresses, yet because the condition or effectiveness of any antecedently applied distresses treatments. it's quite necessary to seek out the probable cause(s) of the pavement failure being investigated. The probable causes area unit commonly explicit, and their area unit usually multiple factors that contributed to the failure. the primary stage in crucial the failure cause(s) is that the fact-finding synthesis, wherever all the data gathered is listed. From this listed data, it's then necessary to see that data supports or refutes every of the attainable failure hypotheses. this might be ab initio done by considering general failure causes, like those associated with construction, materials, design, or the surroundings. it's a lot of needed that specific cause(s) of the failure be thought-about. this can be achieved by hunting attainable failure cause(s) for the failure sort. Once this has been done, it's necessary to see the probable cause(s) of the failure.

3. Results and Analysis

3.1. Visual Observations

The pavement condition index (PCI) methodology provides an evaluation based on visual inspection, namely on the distresses observed on the pavement. The objective of this dissertation is to help reducing its limitations and make it easier and faster to use. The field survey was carried out visually in two stages: by driving a car, and by walking along the road. In the first stage, while the survey team was driving a car along the road at slow speed, observed the affected pavement sections, and carried out ride quality assessment of the pavement surface. This stage of survey is a kind of reconnaissance of the study area. The second stage of survey was carried out by walking through the study surface area closely observed, identified, and record the defects on failed pavement sections. The distresses were observed during the inspection as shown in figure-3

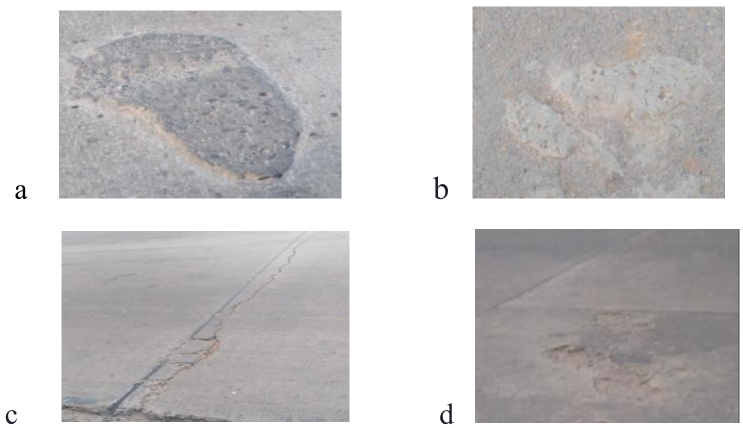

Figure 3: Distresses observed in different bridges

b) Bleeding b) Corrugation c) random Cracking d) settlement

3.2. Pavement Condition Rating

The pavement condition assessment is a very important and significant phase for the purpose of maintenance of any road moreover it can also be used to find the most arising distresses and finding possible solution for these distresses. The first step of the design project is to find the possible distresses and condition of bridge decks so that those same distresses could be replicated in laboratory preparation of the samples. The condition assessment of pavement includes finding the characteristics and different distresses for the evaluation of condition of the pavements. With the passage of time different indicators have been developed for the making decisions about pavement maintenance.

Pavement Condition Index (PCI), Pavement Quality Index (PQI) and Pavement Serviceability Index (PSI) are different indicators. So, in order to identify different issues related to bridge decks a Performa has been prepared using different survey forms provided by different authorities. Federal highway authority provides a Performa to find the serviceability index of the roads similarly AASHTO also provides condition assessment techniques for pavements however the Performa prepared is more like the FHWA Performa for the assessment of the concrete bridge decks. Each of the bridge surveyed will be first identified for its category, date of commissioning and the material used for bonding between the concrete deck and the asphalt overlay. Afterwards the bridges will be surveyed for the defects and distresses using Performa.

Table 1: Analysis of the visual observations

Sr No	Bridges	PCR	served Distresses
1	A	56	Pop outs and multiple cracking
2	B	42	Random cracking
3	C	59	Bleeding was the main distress
4	D	39	Longitudinal and transverse cracking extreme
5	E	85	Minor cracking

The analysis was done by using the figure-1 as reference and by visual observations filling the Performa, we will be able to achieve the pavement condition rating (PCR). The data collected from the field survey of the existing pavement surface failures were analyzed. The most common failures recorded include cracking, rutting, patching, potholes, depressions and corrugation. The measured distresses with the different level of severity are analyzed.

4.Conclusion

To explore the effect of loading on the interfacial bonding between concrete bridge deck and asphalt overlay flexure testing apparatus will be used on small-scale samples that will be prepared in laboratory. The choice of flexure testing is based on the fact that maximum bending occurs on the mid span of the deck and the overlay will de-bond in this mid span which will help us in assessment. The prototype will be a small-scale concrete bridge deck with asphalt overlay on it. The size of the prototype will be kept considering the size of the flexure testing machine.

The thickness of the concrete bridge deck, its reinforcement, mix design and thickness of asphalt overlay, its JMF will be decided such that the sample is a small-scale replica of a real bridge decks with asphalt overlays. The prototype will be tested using the destructive testing which is the direct shear test in which a small-scale bridge deck will be replicated in laboratory and then a core sample will be taken out to subject it to shear test loading. The interfacial behavior of concrete bridge deck and asphalt overlay will be assessed under maximum loading will be then analyzed. The direct shear test is design so as to find the interfacial shear strength of bonded asphalt overlay with concrete. A core sample is taken from the specimen and is then subjected to shear force and compression force simultaneously. The shear speed is varied between 3.5-50mm/min and the compressive force varies from 0.1-0.7mpa.

Acknowledgement

I would like to acknowledge my group members Minhas Shah and Muhammad Ali Tariq. The careful and constructive suggestions by the anonymous reviewers are gratefully acknowledged

References

Abtahi, S. M., Sheikhzadeh, M., & Hejazi, S. M. (2010). Fiber-reinforced asphalt concrete a review Construction and Building Materials, 24(6), 871-877. Alsaif, A., Koutas, L., Bernal, S. A., Guadagnini,

M.,Pilakoutas, K. (2018). Mechanical performance of steel fibre reinforced rubberised concrete for flexible concrete pavements. Construction and Building Materials, 172, 533-543.

Baek, J., & Al-Qadi, I. L. (2006). Finite element method modeling of reflective cracking initiation propagation: Investigation of the effect of steel reinforcement interlayer on retarding reflective cracking in hot-mix asphalt overlay. Transportation Research Record, 1949(1), 32-42.

Bazzaz, M., Darabi, M. K., Little, D. N., & Garg, N. (2018). A straightforward procedure to characterize nonlinear viscoelastic response of asphalt concrete at high temperatures. Transportation Research Record, 2672(28),481-492.

Canestrari, F., & Santagata, E. (2005). Temperature effects on the shear behaviour of tack coat emulsions used in flexible pavements. International Journal of Pavement Engineering, 6(1), 39-46.

Cheng, D., & Hicks, R. G. (2012). Survey on the Use of Rubber Modified Asphalt Pavements in Cold Regions.

Chung, D. D., & Chung, D. (2012). Carbon fiber composites. Elsevier El Halim, A. A., El-Desouky, A., & El Halim, A. (2017, July). Extending the Service Life of Bridges Through Proper Compaction of Asphalt Decks. In International Congress and Exhibition" Sustainable Civil Infrastructures: Innovative Infrastructure Geotechnology" (pp. 49-65). Springer, Cham

Azari, H., & Lin, S. (2020). Evaluation of the Impact Echo Method for Concrete Bridge Decks with Asphalt Overlays. Transportation Research Record, 2673(2), 436 444.

Haynes, M. A., Coleri, E., & Sreedhar, S. (2020). Impermeable Asphalt Concrete Layer to Protect and Seal Concrete Bridge Decks. Transportation Research Record, 0361198119841041.

Hicks, R. G., Dussek, I. J., & Seim, C. (2000). Asphalt surfaces on steel bridge decks. Transportation Research Record, 1740(1), 135-142.

Lin, S., Meng, D., Choi, H., Shams, S., & Azari, H. (2018). Laboratory assessment of nine methods fornondestructive evaluation of concrete bridge decks with overlays. Construction and Building Materials, 188, 966-982.

Rhee, J., Kim, H., Ock, C., & Choi, J. (2018). An investigation of the deterioration characteristics of concrete bridge decks with asphalt concrete in Korea. KSCE Journal of Civil Engineering, 22(2), 613- 621

Zhao, H., Cao, J., & Zheng, Y. (2017). Investigation of the interface bonding between concrete slab and asphalt overlay. Road Materials and Pavement Design, 18(sup3), 109-118.

Zhiyuan, Y., Ji, B., Fu, Z., & Ge, H. (2016). Fatigue performance of a cracked rib-deck welded joint retrofitted by ICR technique. International Journal of Steel Structures, 16(3), 735 -742.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

COMPARATIVE STUDY OF CONCRETE PROPERTIES USING DIFFERENT COARSE AND FINE AGGREGATE ABUNDANTLY AVAILABLE IN PAKISTAN

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Abstract
This research work includes the study of physical properties of some natural aggregates witch are available in Pakistan. This study is of considerable importance since aggregates are mostly used in concrete and road construction. With the passage of time concrete structure are becoming more and more common and at the same time, study of the property of aggregates is also becoming essential because it imparts many properties to concrete.
They are many deposits of aggregates, in Pakistan at various places but a few of them are established quarries. The Remaining deposits required study of their suitability. If, before taking in hand a project involving the use of aggregates, various properties of aggregates available from various quarries are studied, it becomes easy to decide which aggregate is going to be the most suitable and economical for the particular case. Firstly, materials used in this research have been identified, located and transported. These were then followed by physical properties of these selected materials. In total two types of fine and two types of coarse aggregates were used namely Lawrencepur, Chenab Sand and two types of coarse aggregates Margalla and Sargodha crush.
In this project, various properties of aggregate from various sources are studied with best accuracy and then their results are compared with the standard results. The information's regarding behavior of various aggregates are given, which can be successfully employed for further works.
The research focused on variation in compressive and flexural strength of concrete due to different types of fine and coarse aggregates. Casting of all simples was done according to ASTM standards. Simple after casting were tested for compressive and flexural strength after 07, 14 & 28 days. The research has related that apart from various other factors, effective water cement ratio, which also depends on water absorption of aggregates.

ASTM: American Standards for Testing and Materials,
FW: Future Works ,LWS:Lawrencepursand. CS: Chenab sand , W/C: Water/Cement Ratio

1. INTRODUCTION

1.1 General

There are many types of materials in the construction. The most commonly used materials are steel, cement, concrete, bricks, stone, clay etc. But concrete is one of the most widely used materials in the construction industry. After water concrete is most widely using material. Globally 11 billion tons of concrete is consumed each year. Concrete is the most common construction material in the world because it has very good durability, mechanical properties, workability and relative low cost. It is suitable almost for all type of structures and all type of environments like building s, bridges etc. Since it can be mould into any desired shape which is also a versatile property of concrete. Concrete s stone like material obtained by mixing of cement, sand and gravel, or other aggregates with water. It can be poured into any desired shape and dimension of structure through a chemical reaction know as hydration the paste hardens and gain strength to form concrete. As long as the concrete kept cured it continues to gain the strength because the process of hydration continues in the presence of water. Mix water in concrete can be utilized for hydration, if it is secured by curing. Generally concrete gains most of its strength within 28 days but a slower process of hydration continues for many years. Concrete is a brittle material and is very strong in compression but weak in tension.

Natural aggregates, are fragments of rocks due to disintegration or weathering action, or other material of mineral composition. In the past, aggregate was viewed as a mere inert material and its extensive use was considered due to its cheapness and abundance. In fact, aggregate is not truly inert and its physical, thermal and sometime also chemical properties influence its performance.

Here we make a humble contribution in understanding those basic properties of aggregate which are so important to know to predict the behavior of aggregate under various conditions of usage.
Both the coarse and fine locally available aggregates are in abundance and are being constantly used for huge and small construction projects in private as well as in public sector.

Apart from exploring effects of two locally available fine aggregates, i.e., Lawrencepur and Chenab sands, effects of blended sands on strength and workability have also been studied by mixing two fine aggregates of Lawrencepur and Chenab sands in equal proportions. Two types of sands, i.e., Lawrencepur and Chenab collected and used individually. Lawrencepur sand was obtained from a local supplier, whereas Chenab sand was collected from source. Chenab sand was obtained from River Chenab near Wazirabad. However, Margalla Crushed Stone was used as coarse aggregate and 0.5 water cement ratio were kept unchanged. The aggregates were used in day conditions.

For determining concrete strength, standard 6" ×12" cylinders and 4" × 4" × 20" Beam used as prescribed by the ACI code. Routine traditional mixes have been used to make study more meaningful for application point of view and for making the results available for these two most commonly used concrete mixes.

2. METHODOLOGY

In this research work, discussions will made on material physical properties, characteristics, test methods, test standards, concrete mix design and the experimental program for carrying out tests on fresh and hardened concrete.

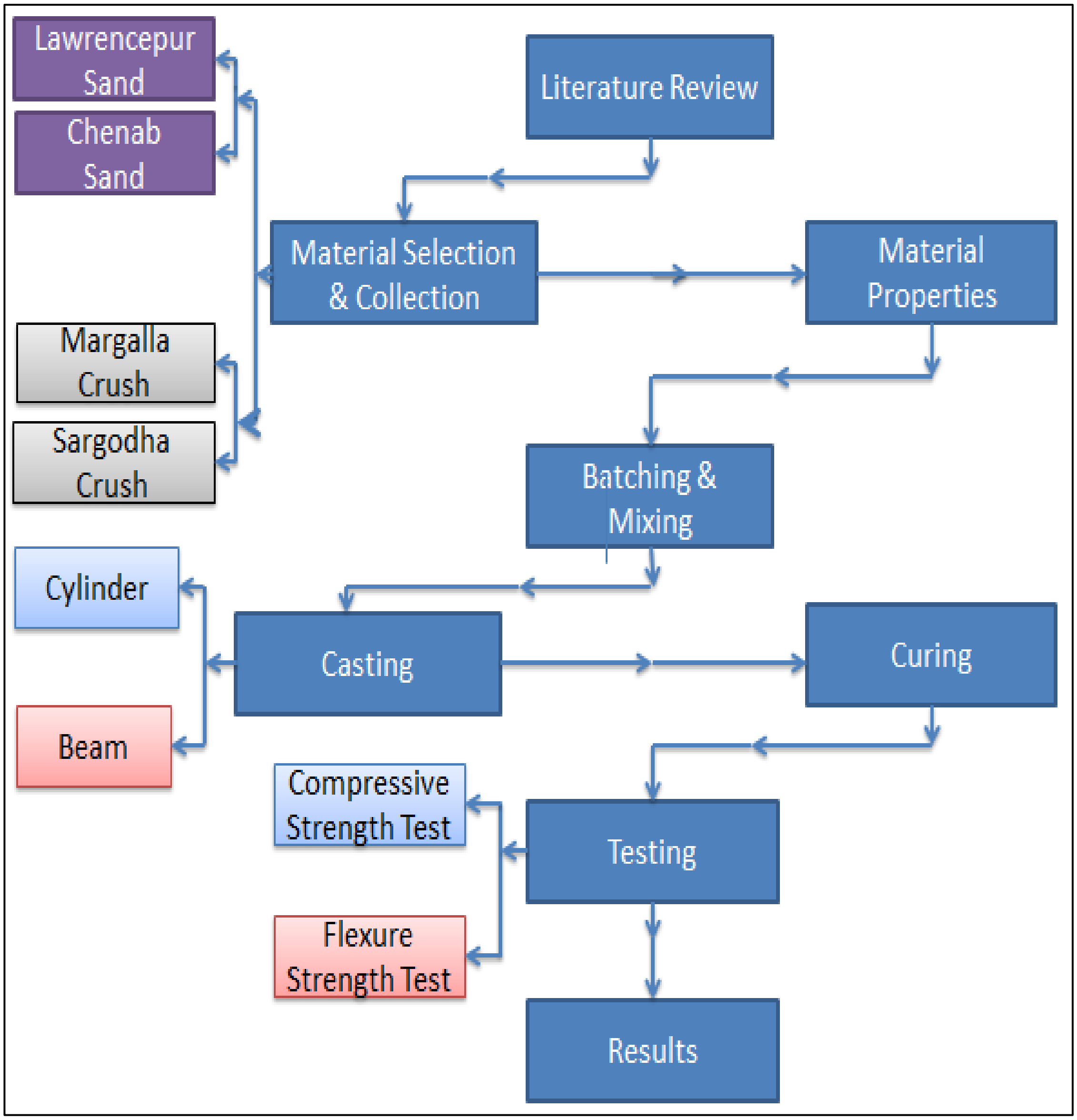

Figure No 1: Research Methodology Flow Chart

3. Experimental Work

3.1 Test Standard

All tests were carried out under the following guidelines of ASTM standards;

3.1.1 Cement Test

Results of tests carried out on cement

- Initial setting time (minutes) = 31
- Final setting time (minutes) = 130
- Consistency (%) = 32

3.1.2 Fine Aggregates Test (Lawerancepur sand)

Results of tests carried out on lawrencepur sand

- Bulk Specific Gravity (Oven Dry) = 2.52
- Bulk Specific Gravity (SSD Condition) = 2.58
- Apparent Specific Gravity = 2.69
- Water Absorption (%) = 2.48
- Moisture Content (%) = 1.02
- Fineness modulus = 2.64

3.1.3 Coarse Aggregates Test (Mar gala crush)

Results of tests carried out on coarse aggregates (Mar gala crush)

- Bulk Specific Gravity (Oven Dry) = 2.63
- Bulk Specific Gravity (SSD Condition) = 2.64
- Apparent Specific Gravity = 2.66
- Water Absorption (%) = 0.546
- Fineness modulus = 7.96
- Bulk Density (Compacted) lbs/ft^3 = 96.76
- Bulk Density (Loose) lbs/ft^3 = 88.16
- Flakiness index (%) = 5.10
- Elongation index (%) = 13.72

3.2.MIXES COMBINATIONS

i. 100% Sargodha + 100% Lawrencepur sands
ii. 100% Margalla + 100% Lawrencepur sands
iii. 100% Sargodha + 100% Chenab sands
iv. 100% Margalla + 100% Chenab sands
v. 100% Sargodha + 50% Chenab + 50% Lawrencepur sands
vi. 100% Margalla + 50% Chenab + 50% Lawrencepur sands

3.3 Test Performed on Concrete

- Slump test
- Compressive strength test
- Flexural Strength Test

3.3.1 Slump Test

Workability of concrete mixture is measured by slump test: The slump test was conducted in accordance with the ASTM C-143 guidelines. In this test the slump cone was used. Three equal layers of concrete were filled in the sliding cone and compressed using 25 strokes of crimping rod. The rod was tempered having a diameter of 5/8in and length of 24 in. The slump test provides a good estimate of expected operability.

Figure No 2: Slump test

Table No.1: Result of Slump Test

Sr. No.	w/c	Mix Proportions	Coarse Aggregate	Fine Aggregate	Compacting Factor	Slump Value (mm)
1	0.5	1:2:4	M	L	0.88	72
2	0.5		S	L	0.91	78
3	0.5		M	C	0.90	76
4	0.5		S	C	0.92	86
5	0.5		M	L+C	0.89	78
6	0.5		S	L+C	0.90	82

3.3.2 COMPRESSIVE STRENGTH TEST

Compressive strength test was performed according to the provision of ASTM C 39. Cylindrical specimens measuring 0.6 inch x 12 inch, were prepared from each batch of concrete and were tested at 28 days of concrete age. The specimens were cured in lab curing tank until the age of testing.
The three specimens were tested in order to obtain an average value.
Figure shows the cylinder placed in calibrated automatic compression testing machine and ready to test. (The compressive strength was calculated by P/A; where "P" is applied load and "A" is area of cylinder).
Cylinder specimens were caped and tested in dry state. Rate of loading and general testing procedure was closely followed as described in ASTM C-39.

Figure No. 3: Compression Testing

Table No.2: Results of Compressive Strength

Sr. No.	Batch No.	Mix Proportions	Coarse Aggregate	Fine Aggregate	Specimen Number	Strength Measured (psi)			Average Strength (psi)		
						7 days	14 days	28 days	7 days	14 days	28 days
1	1	1:2:4	M	L	1	2088	2257	2370	2120	2241	2410.5
2			M	L	2	2152	2225	2451			
3	2		S	L	1	2008	2190	2359	2024	2209	2383
4			S	L	2	2040	2228	2407			
5	3		S	C	1	1664	2220	2960	1734	2213.5	3000
6			S	C	2	1804	2207	3040			
7	4		M	C	1	2052	2640	3144	2037.5	2656.5	3162
8			M	C	2	2023	2673	3180			
9	5		M	L+C	1	1800	2254	2796	1754.5	2283	2829
10			M	L+C	2	1709	2312	2862			
11	6		S	L+C	1	1966	2580	2840	2068	2604	2900
12			S	L+C	2	2170	2628	2960			

For each type two cylinders were cast therefore the tests results have been reported above as per strength obtained for each individual cylinder along with average strengths calculated for each batch at different ages.

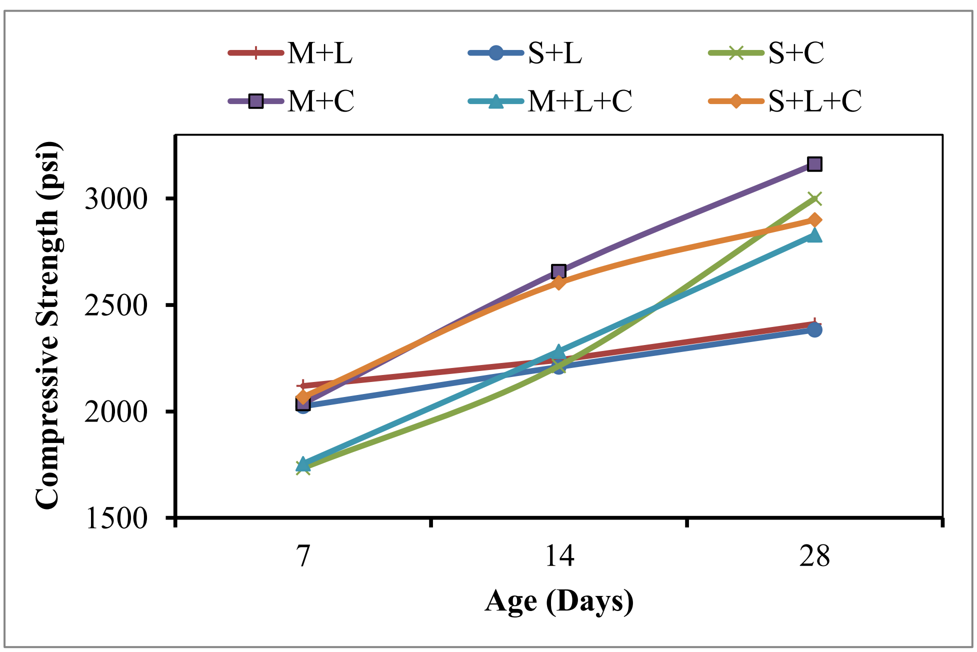

*Figure No. 4: Graphical Representation **of** Compressive Strength*

3.3.3 FLEXURAL STRENGTH TEST

Flexural strength test was performed according to the provision of ASTM C 78. Beam specimens measuring 100 mm x 100 mm x 500 mm were prepared from each batch of concrete and were tested at 28 days of concrete age. The specimens were cured in lab curing tank until the age of testing. The three specimens were tested in order to obtain an average value. The beam specimens were tested in dry state. Rate of loading and general testing procedure was closely followed as described in ASTM C 78. At the end of each test the specimens were broken and observations of concrete matrix were made

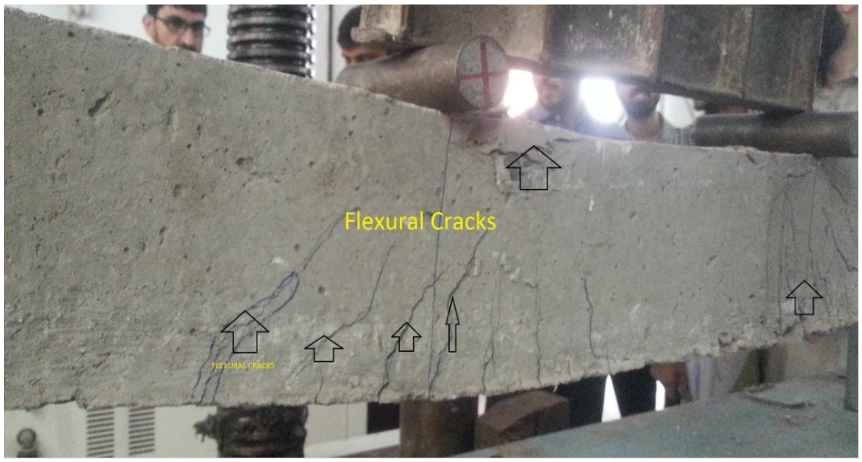

Figure No.4: Flexural Crack

Figure No. 5: Modulus of Rapture

Table No. 4 Results Of Flexural Strength

Sr. No.	Batch No.	Mix Proportions	Coarse Aggregate	Fine Aggregate	Specimen Number	Strength Measured (psi)			Average Strength (psi)		
						7 days	14 days	28 days	7 days	14 days	28 days
1	1	1:2:4	M	L	1	431	530	690	425.5	545	676.5
2			M	L	2	420	560	663			
3	2		S	L	1	430	508	610	419	499	625
4			S	L	2	408	490	640			
5	3		S	C	1	530	590	630	501	600	644.5
6			S	C	2	472	610	649			
7	4		M	C	1	606	680	710	601	673.5	722
8			M	C	2	596	667	734			
9	5		M	L+C	1	540	590	730	550	583.5	709.5
10			M	L+C	2	560	577	689			
11	6		S	L+C	1	530	607	710	505	623.5	695
12			S	L+C	2	480	640	680			

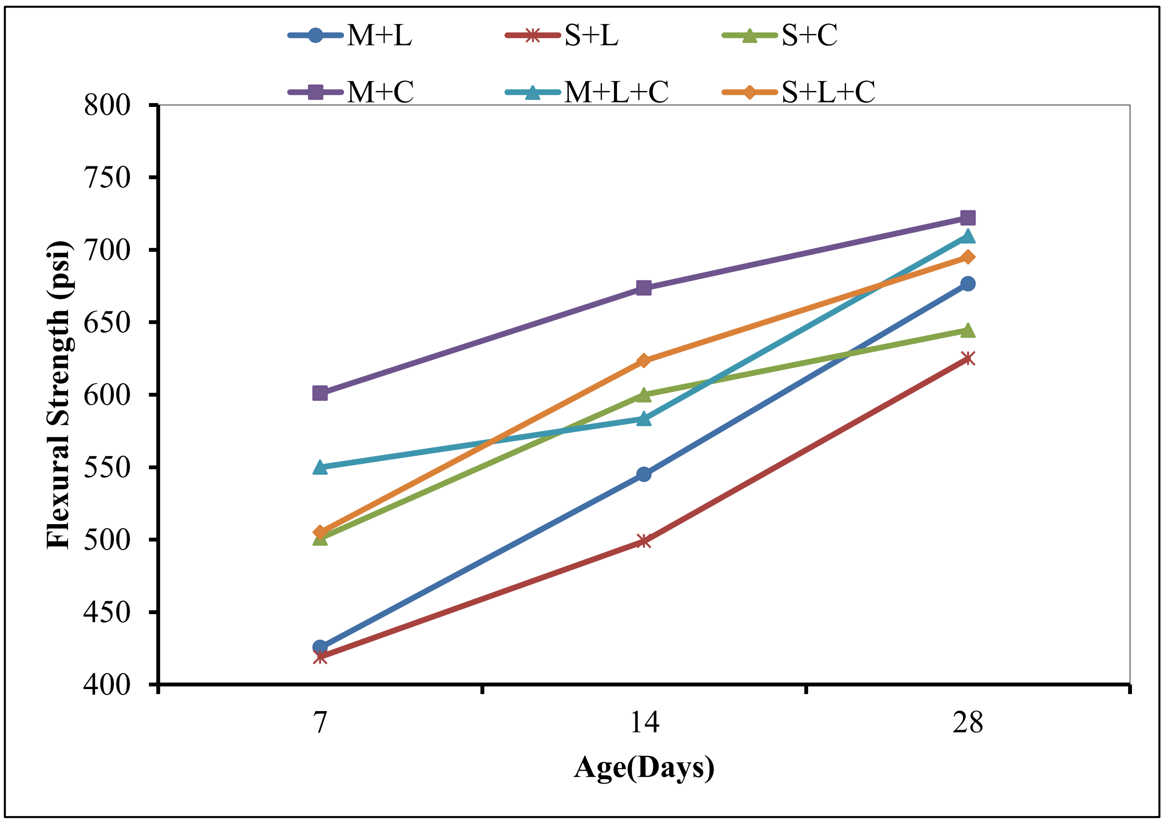

Figure No.7: Graphical Representation of Flexural Strength

4. REFERENCES

1. Akcaoglu, T., M. Tokyy and T. Celik. Effect of coarse aggregate size and matrix quality of ITZ and failure behavior of concrete under uniaxial compression. Cement and Concrete Comp.,26(6); (2004)
2. AyubM., Q. Ali, K. Shahzada, A. Naseer. M. Shoaib and U.Ayub. Engineering Assessment of coarse aggregate used in Peshawar, Int. J. Advanced Structures and Geotechnical Eng., 01 (2); (2012)
3. Basheer, L.,P. Basheer and A. Long. Influence of coarse aggregate on the permeation, durability and the microstructure characteristic of ordinary Portland Cement Concrete. Construction and Building materials, 19 (9); (2005)
4. Elsharif A., M. D. Cohen and J. Olek. Influence of aggregate size, water cement ratio and age on the Micro Structure of the interfacial transition zone. Cement and Concrete Res., 33(11); (2003)
5. Zahid Ahmad Siddiqi; Concrete Technology (Part 1); 2009. University of Engineering and Technology, Lahore.
6. Kosmatka S., Kerkhoff B., Panarese W. Aggregate for concrete, Design and control of Concrete Mixture. 14th Ed. Portland cement Association, 2003, USA.
7. Abdullahi M. Effect of Aggregate type on Compressive strength of concrete. International Journal of Civil and Structure Engineering, Volume. 2, No 3, 2012
8. Naville A. Property of Concrete. 2nd Ed. Pearson education limited. London, 2010.
9. Tesong W., Lynsdale C., Cripps J. Aggregate cement paste interface. Influence of aggregate physical properties. Cement and concrete Research, Vol. 28, No. 10, 1998
10. Kaplan M. Flexural and compressive strength of concrete as effected the property of course aggregate. Journal American Concrete Institute, Vol. 55, 1959
11. Aitcin, P.C.; 2003, The durability of high performance concrete: a review, cement and concrete composition, 25(4-5): 409-420.
12. Nawy, E.G; Fundamental of high performance concrete, 2nd edition, New York Publisher: John Wiley & Sons, 2001

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

SOIL REINFORCEMENT BY USING GEOTEXTILE

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Abstract
In many activities concerned with the use of soil, the physical properties like Stiffness, Compressibility and Strength are some of the few important parameters to be considered of the many methods involved in improvement of soil properties, soil reinforcement is method concerned with increase of strength properties of soil. In soil reinforcement, the reinforcements or resisting element are of different materials and of various forms depending upon the intended use. The reinforcement can be provided permanently or temporarily to increase strength of adjacent structures. The present topic of discussion involves different materials, forms and applications of soil reinforcement.
Keywords: GET: Geo technical , FR: Fiber-reinforced ,PP: Physical properties ,SP: Soil properties ,TS: Tensile stress ,IF: Internal friction

1. INTRODUCTION

1.1 GENERAL

Soil reinforcement is a technique to improve the engineering characteristics of soil. In this way, using natural fibers to reinforce soil is an old and ancient idea. Consequently, randomly distributed fiber-reinforced soils have recently attracted increasing attention in geotechnical engineering for the second time. Reinforced earth is a combination of earth and linear reinforcing strips that are capable of bearing large tensile stresses. The reinforcement provided by these strips enable the mass to resist the tension in a way which the earth alone could not. The source of this resistance to tension is the internal friction of soil, because the stresses that are created within the mass are transferred from soil to the reinforcement strips by friction

Soil reinforcement, which involves the use of formation in a soil mass to improve its mechanical properties, has become a widely used earthwork construction method that provides technically attractive and cost-effective grade separations at the ground surface. Reinforced soil walls generally provide vertical grade separations at a lower cost than do traditional cast-in-place concrete construction

Basic principles of soil reinforcement already existing in nature and are demonstrated by animals, plants and birds. The modern form of the soil reinforcement was first applied by Vidal (1969). Based on the Vidal's concept the interaction between soil and the reinforcing horizontal member is solely by friction generated by gravity. Applying this concept retaining walls were built in France in 1986 (Prashant Patil). Nowadays this technique is widely used in Europe and U.S.A. This technique is yet to become popular in India and Pakistan, and the constraining factor being identified as the non-availability of fiber and cost of reinforcing material. Reinforced soil is somewhat analogous to the reinforced concrete. But direct comparison between the functions of reinforcement in the two cases is not valid.

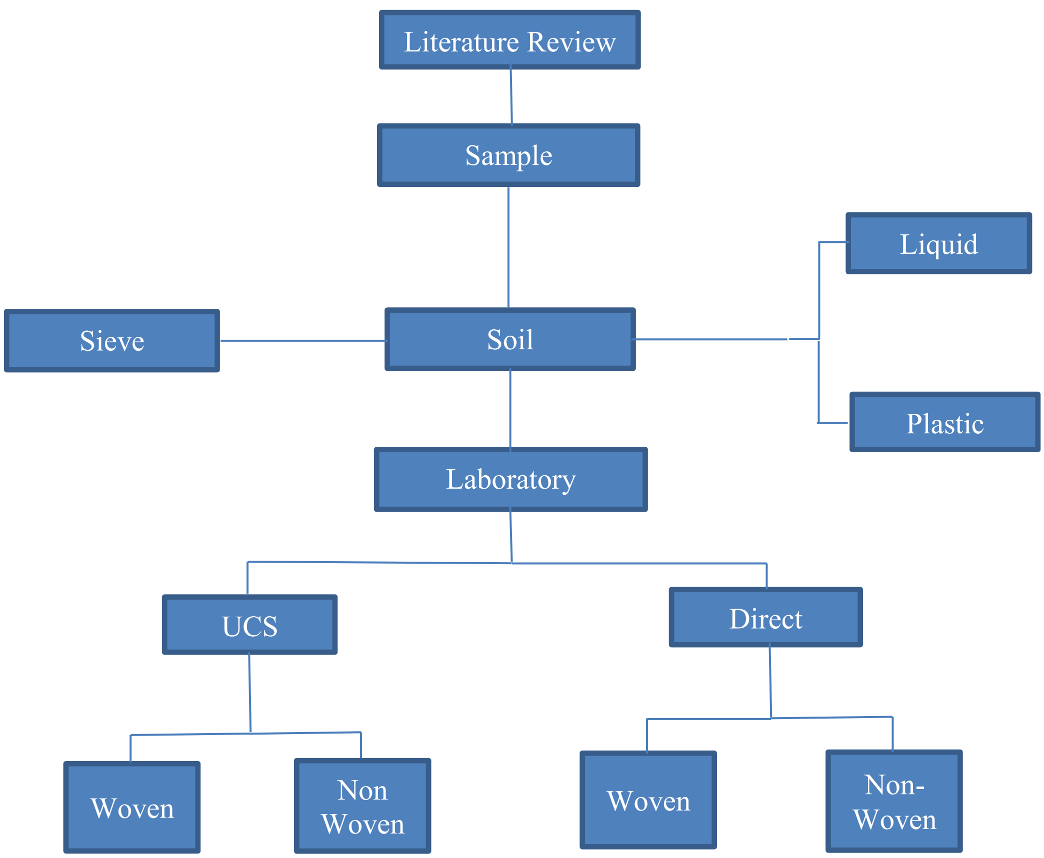
The mode of action of reinforcement in soil is not one of carrying the developed tensile stresses as in reinforced concrete but of anisotropic reduction of normal strain rate.

Geotextile is the material which is use in soil reinforcement and it is determined by the method used to combine the filaments or tapes into the planar textile structure. The vast majority of geotextiles are either woven or nonwoven. Woven geotextiles are made of monofilament, multifilament, or fibrillated yarns, or of slit films and tapes.

Although the weaving process is very old, nonwoven textile manufacture is a modem industrial development. Synthetic polymer fibers or filaments are continuously extruded and spun, blown or otherwise laid onto a moving belt. Then the mass of filaments or fibers are either needle punched, in which the filaments are mechanically entangled by a series of small needles, or heat bonded, in which the fibers are welded together by heat and/or pressure at their points of contact in the nonwoven mass. Stiff geogrids with integral junctions are manufactured by extruding and orienting sheets of polyolefins. Flexible geogrids are made of polyester yarns joined at the crossover points by knitting or weaving, and coated with a polymer.

1.2 RESEARCH METHADOLOGY

At first we have reviewed the literature to familiarize ourselves with latest techniques of soil reinforcement. Then we visit the site and collect the soil sample.
In the laboratory 1st of all we classified the type of soil by performing the sieve analysis, liquid limit and plastic limit tests. After classification of soil we perform direct shear test and UCS test to find shear strength and internal friction angle of soil with reinforcement and without reinforcement material.



FigureNo. 35: Research Methodology Flow Chart

2. RESULTS AND DISCUSSION

2.1 GENERAL

The effect of soil reinforcement by woven and non-woven geotextile was determined by the help of lab tests including direct shear test and unconfined compression strength test.
The disturbed sample was collected and lab tests were conducted according to relevant ASTM standards.

2.1.1 MOISTURE CONTENT

Table No. 1: Results of Moisture Content Test

Serial	W_1	W_2	W_3	W%
1	10.71	33.19	29.91	17.08
2	12.24	34.9	31.37	18.45
3	12.13	43.51	38.78	17.75

2.1.2 SIEVE ANALYSIS

Table No 2: Results of Sieve Analysis

Sr. No	Sieve No.	Sieve Opening (mm)	Weight Retain (gm)	% Retain	Commulative %Retain	% Passing
1	#4	4.75	0	0	0	100
2	#8	2.36	16	1.63	1.63	98.37
3	#10	2	14	1.42	3.05	96.95
4	#16	1.18	32	3.25	6.3	93.7
5	#40	0.425	64	6.52	12.82	87.18
6	#100	0.15	680	67.2	80.02	19.98
7	#200	0.075	164	16.7	96.72	3.28
8	Pan		30	3.05	99.77	0.23

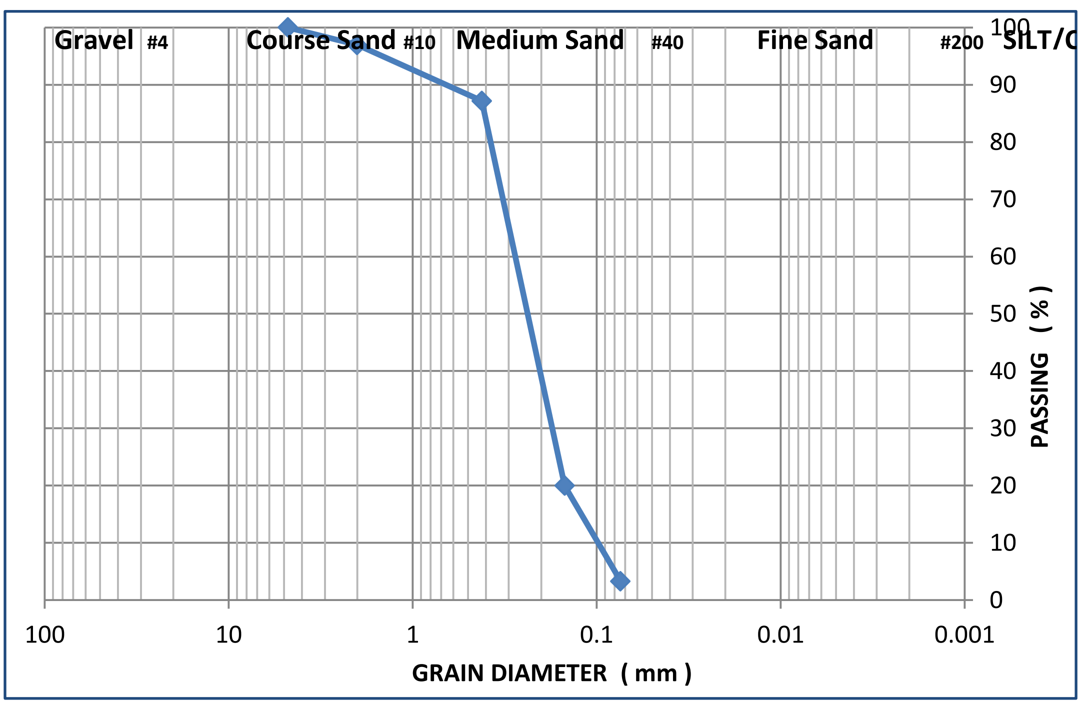


Figure No. 2. Sieve Analysis

2.2 ATTERBERG LIMITS

Table No.3: Results of liquid limit

Sample codes	W_1	W_2	W_3	(w2-w3/w3-w1) *100	N	Liquid limit
Ce-ge\0206	10.71	33.19	29.91	17.08	29	17.393
Ce-ge\0202	12.24	34.9	31.37	18.45	20	17.96
Ce-te\1902	12.13	43.51	38.78	17.75	25	17.75
					Average	17.70

2.3 USCS AND AASHTO CLASSIFICATION OF SOIL

Standard Test Reference
ASTM D 422 - Standard Test Method for Particle-Size Analysis of Soils
Weight of Dry Soil Sample = W = 1000 gm

Table No 5: Soil Classification

Sieve No.	Sieve Size (mm)	Wt. Retained (gm)	% Wt. Retained	Cumm. % Wt Retained	%age Passing
4	4.750	0	0	0	100
8	2.36	16	1.63	1.63	98.37
10	2.000	14	1.42	3.05	96.95
16	1.18	32	3.25	6.3	93.7
40	0.425	64	6.52	12.82	87.18
100	0.150	680	67.2	80.02	19.98
200	0.075	164	16.7	96.72	3.28
Pan	-	30	3.05	99.77	0.23

Percentage of Various Fractions:

Sandy Soil = 96.72%

Silty Soil = 3.28%

Unified Soil Classification System (ASTM – D2487) Based on the Material Passing 75 mm (3in) Sieve					
$F_{200} < 50$	R_4 / R_{200}	Fines = F_{200}	Cu	Cz	Group Symbol
3.28	0 < 0.5	$3.28 \leq 5$	2.25	0.891	**SP**

- *Retaining on Sieve No. 4* $R_4 = 0$
- *Retaining on Sieve No. 200* $R_{200} = 16.7$
- $D_{60} = 0.27$

- D_{30} = 0.17
- D_{10} = 0.12
- Group Name = Poorly Graded Sand

AASHTO Classification (ASTM – D3282)			
% Passing # 200	Liquid limit	Plasticity Index	Classification
3.28 < 35	17.70<40	0	**A-3**

2.4 DIRECT SHEAR TEST

Table No 6: Direct shear test results

Direct shear Test with woven Geotextile					
Layers	0	1	2	4	6
ϕ	28	30	32	33	32
Shear Strength	21.84	24.14	27.2	30.28	29.3
Direct shear Test with non- woven Geotextile					
Layers	0	1	2	4	6
ϕ	28	30	32	35	33
Shear Strength	21.84	25.1	27.25	32.24	31.31

2.5 UNCONFINED COMPRESSION TEST

Table No 7: Results of Un-confined Compression Test

Unconfined Compression Test By non-woven Geotextile	
Soil Layers	Shear Strength
Simple Soil	66.4465004
1 Layer	109.6647895
2 Layers	113.9013475
3 Layers	114.2413516

2.6 COMPARISON OF DIRECT SHEAR TEST

Woven Geotextile Comparison

Table No. 8: Comparison between woven Geotextile layers Soil Reinforcement

Direct Shear Test with woven Geotextile					
Layers	0	1	2	4	6
ϕ	28	30	32	33	32
Shear Strength	21.84	24.14	27.2	30.28	31.3

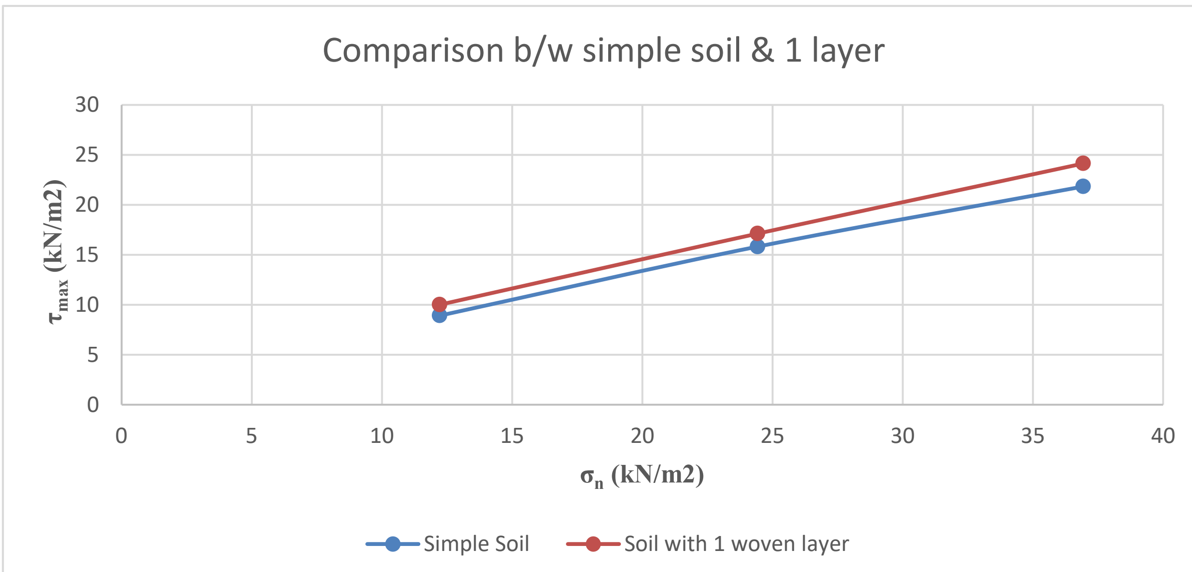


Figure 3: Comparison between simple soil and soil with 1 woven layer

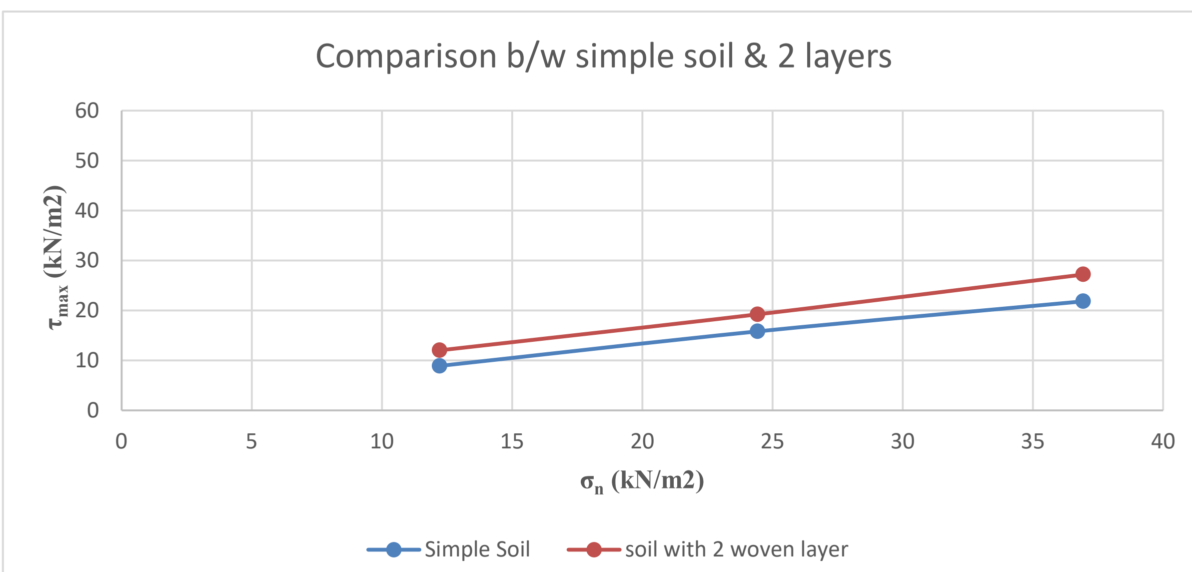


Figure 4.: Comparison between simple soil and soil with 2 woven layers

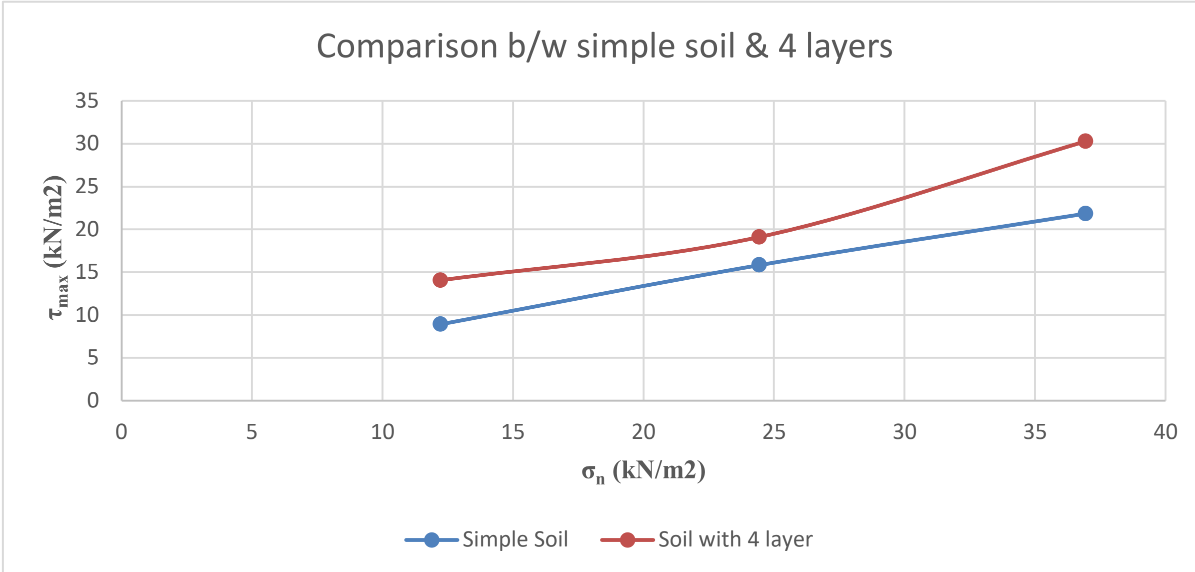


Figure 5: Comparison between simple soil and soil with 4 woven layers

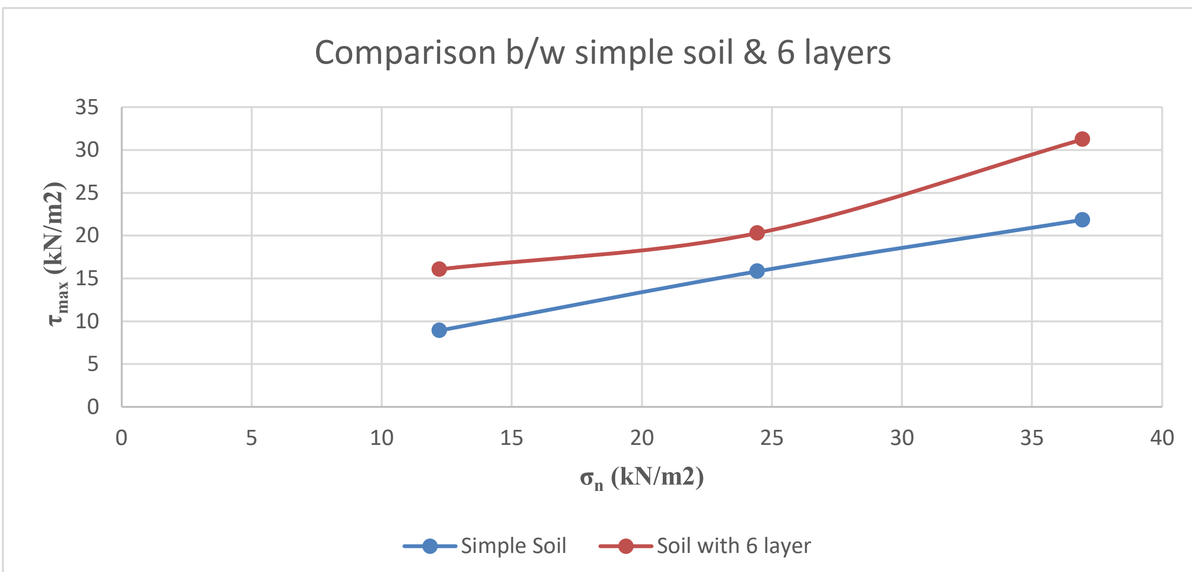


Figure 6: Comparison between simple soil and soil with 6 woven layers

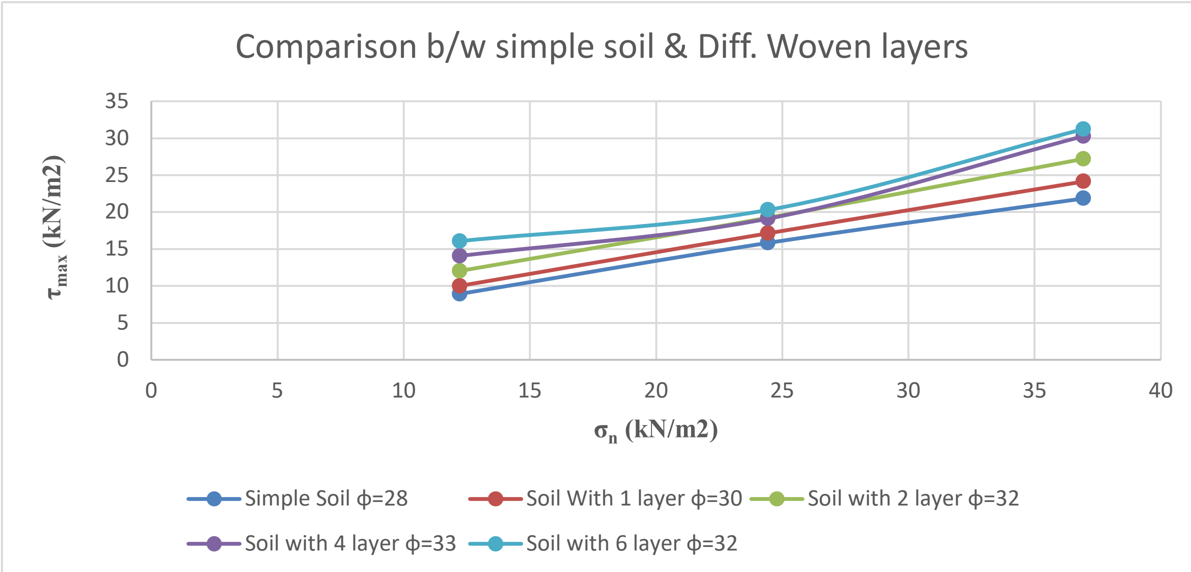


Figure 7: Comparison between simple soil and soil with different woven layers

3. REFERENCES

A. T. (2016, 09). Retrieved from Business and agriculture: https://www.in.gov/indot/div/mt/aashto/testmethods/aashto_t27.

Caramoni, F. a. (1986). Caracterização físico-hídrica de um latossolo roxo distrófico do município de Londrina, PR. *Pesquisa agropecuria brasileira*.

constructor, t. (2018, january). *Geogrids- Types, Functions, Applications and Advantages in Construction*. Retrieved from the constructor civil engineering home: https://theconstructor.org/building/geogrids-types-functions-applications-advantages/15190/

Coquet. (1997). Water movement and soil swelling in a dry, cracked Vertisol. *science direct*, 113-123.

Davidc. (2017, 08 15). *Commercial Construction and renovation*. Retrieved from http://www.ccr-mag.com/what-is-soil-reinforcement-and-how-is-it-done/

Ebo, O. a. (1995). The effects of organic and inorganic amendments on soil physical properties and maize production in a severely degraded sandy soil in southern Nigeria. In O. a. Ebo, *Bioresource Technology* (pp. 117-123).

Gofar, N. (2008). Geosynthetic reinforced retaining structures. 30.

K.P.Panayiotopoulos, C. (1984). Compaction and shrinkage of sands and sand-kaolin

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

TREATMENT OF EXPENSIVE SOIL BY USING SILICA FUME

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Abstract

This study encompasses an experimental investigation and dissemination of the findings of stabilization of an expansive soil obtained from Nandipur, Punjab by using silica fume (SF). The light buildings are continuously subjected to structural distresses, hence repair and maintenance due to cyclic swelling-drying of the underlying expansive soils induced by climatic wetting and drying. There is a growing interest in utilizing industrial waste mainly for the mitigation of swelling-shrinking soils.
Within the scope of this study, primarily the stabilization materials were tested for their suitability to be used as pozzolans. After which indicated silica fume (SF) are sufficiently active, it was used in 5% SF, 10% SF, 15% SF proportions by dry mass of the soil. Physical properties and volume change behavior of the natural soil and treated soil were studied. After completing the experimental program which included the grain size distribution, atterberg limit, specific gravity, moisture and dry density relationship, unconfined compressive strength (UCS) and swell potential. It was concluded that there has been a notable mitigation mainly in the swelling and shrinkage behavior.
Finally, it was concluded that 5% SF addition has been more effective. Therefore, it is recommended to be mixed and compacted with the expansive soils subject to light loads, such as pavements, roads, and at most two story buildings.
Keywords: SF: Silica Fume, UCS: unconfined compressive strength ,AL: Atterberg limit MC: Moisture Content ,SSS: swelling-shrinking soils

1. INTRODUCTION

1.1 General

Expansive soil, which is also called shrink-swell soil, is the main cause of foundation problems. Depending on the amount of moisture in the ground, these soils will experience changes in volume. Such soils when used as foundation, can cause lifting of structures when subjected to high moisture, while desiccation causes shrinkage, hence settlement of buildings.
Problem of expansive soils has appeared as cracking and break-up of pavements, railways, highway embankments, roadways, building foundations, slab-on-grade members and, channel and reservoir linings, irrigation systems, water lines, sewer lines. (Kehew, 1995). Therefore, if the moisture content of these soils can be stabilized, problems related to foundations can be often alleviated. There are several types of stabilizers used for solving this problem. One of the bio-waste stabilizers is rice husk ash (RHA). Rice husk ash is an agricultural waste, taken from rice mills after breakdown of the rice from paddy. It is often used as a fuel in the boiler furnaces in sugar and paper industries for producing steam.
It is reported that damage to the structures due to expansive soils has been the most costly natural hazard in some countries. In the United States damage caused by expansive clays exceeds the combined average annual damage from floods, hurricanes, earthquakes, and tornadoes (Jones and Holtz, 1973).
Developing countries like India, Bangladesh, Malaysia and Iran, which have plenty of rice, produce large amounts of RHA annually. Main properties of RHA are first, having good adsorption characteristics

(because of small particle size and large specific area). RHA is a good pozzolanic material, which can bind clay particles together and reduces the adsorption of water by clay particles, which reduces the swelling properties of expansive soils and improves their volume changes. Second property is having low cost and locally available.
According to the unpleasant properties of expansive soils, silica fume (SF) is chosen. Silica fume is available in the form of an ultrafine waste, with high specific surface. It is a by-product of silicon.

2. METHODOLOGY

Methodology of this research work consists of the soil classification, characterization of Atterberg's limits, unconfined compressive strength, compaction characteristics and swell potential of expansive soil. Laboratory testing was carried out in four phases. These phases are listed below:

1) Characterization of Untreated Soil
2) Properties of Soil treated with Silica Fume at various dosages of 5%, 10%, 15% and 20%.

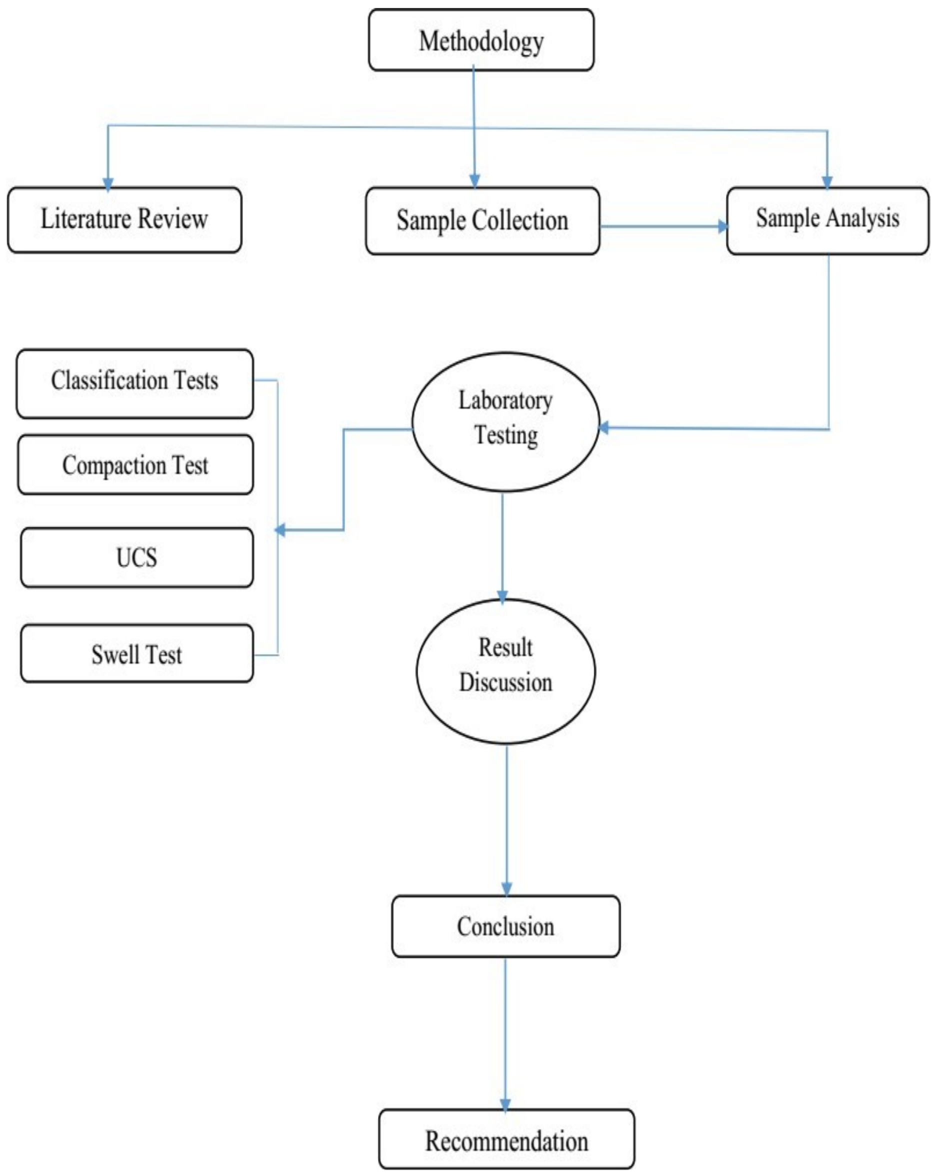

Figure. 1: Process of research being carried out shown by Flow Chart

3. RESULTS AND DISCUSSIONS

Table No. .1: Test results of particle size analysis of pure soil and additives

Samples	Sand (%)	Silt size particles (%)	Clay size particles (%)
Pure Soil	8	26	59
Silica Fume	0	69	23

The particle size analysis of pure soil shows the percentage of material as 9 % sand, 28% silt and 63% clay, as the silica fume was partially replaced in black cotton soil by 5%, 10%, 15% and 20%, it was experimentally analyzed, there is a decrease in the percentage of clay size particles ranging from 63% to 49% and increase in the percentage of silt size particles ranging from 28% to 41%.

Table 4.2 shows the percentage of particles present in the pure soil and treated and soil with silica fume.

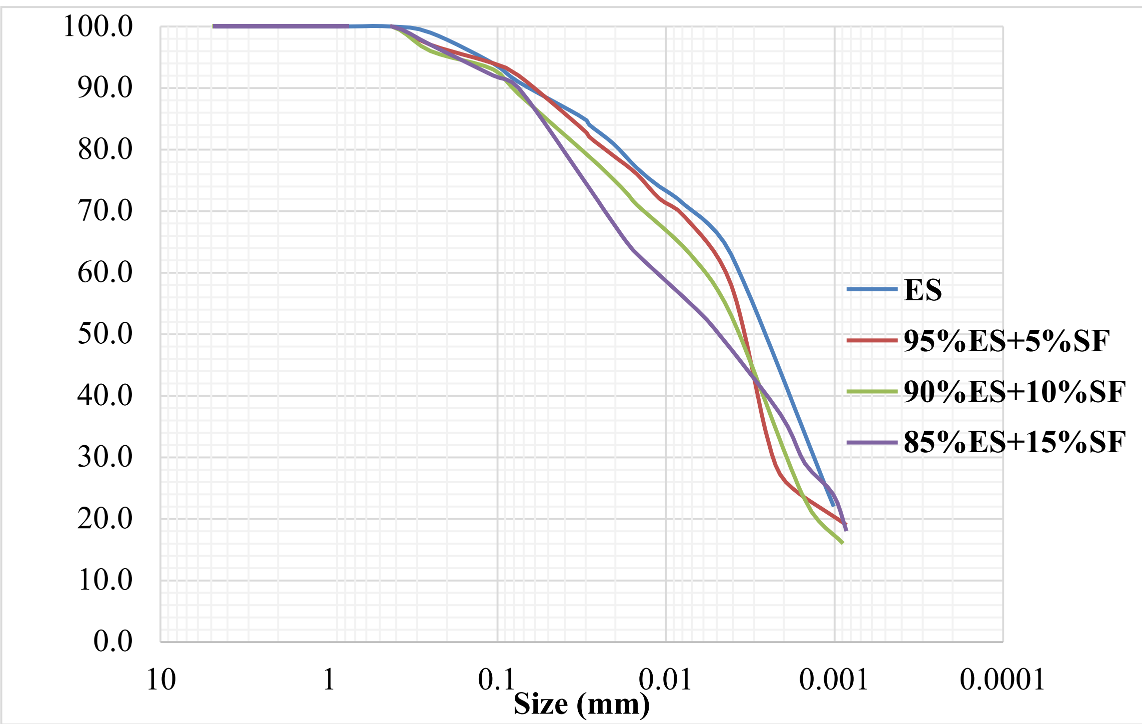

Figure No. 2: Gradation curve of silica fume mixed soil and pure soil

Table No. 2: Test results of particle size analysis pure soil and silica fume treated soil

Samples	Sand (%)	Silt size particles (%)	Clay size particles (%)
Pure Soil	8	26	59
95% ES + 5% SF	7	32	55
90% ES + 10% SF	10	33	50
85% ES + 15% SF	9	37	48

Table No 3: Test results of specific gravity of pure soil and treated with additives

Additives Percentage	Silica Fume
0%	2.710
5%	2.683
10%	2.632
15%	2.611

3.1.3 ATTERBERG LIMITS

Casagrande apparatus was used to evaluate the liquid limit of untreated soil, while threads of 1/8 inch were made to check the plastic limit of soil. These tests were performed in accordance with the ASTM D4318-05 standard. Test results of untreated soil shows values of liquid limit is 45.81%, plastic limit is 36.63% and plasticity index is 25.05%.

Figure 4.3 shows a relationship of liquid limit and plastic limit with increasing percentage of silica fume being partially replaced in black cotton soil and test results are shown in table 4.6, when silica fume was partially replaced in black cotton or expansive soil, there is an appreciable increase in liquid limit of treated soil ranging from 45.81% to 47.40%, plastic limit is reduced ranging from 36.63% to 36.01% and plasticity index is increased from 25.05% to 29.05%

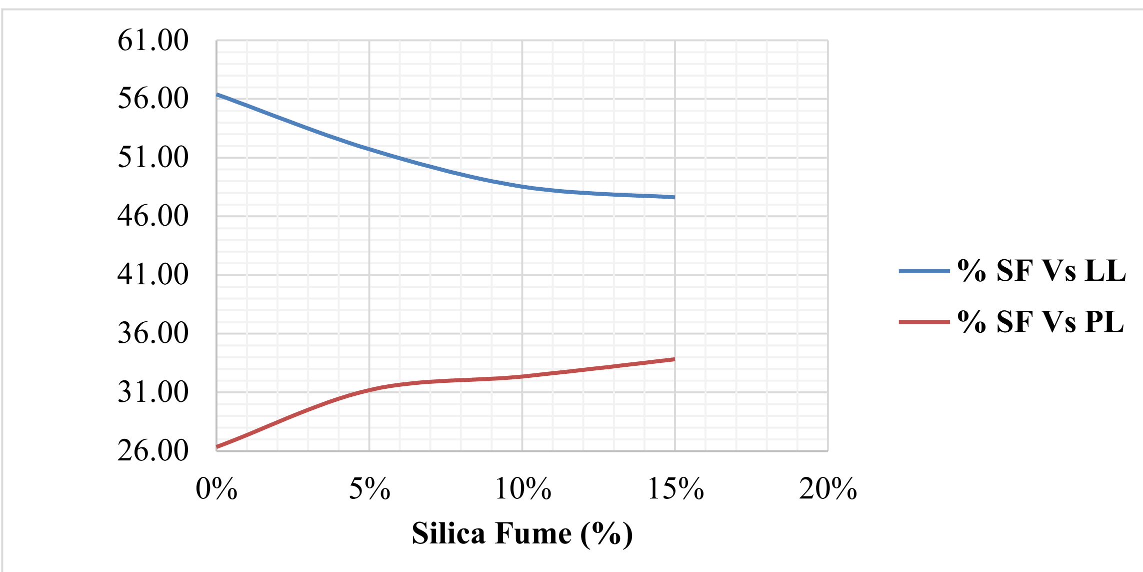

Figure No. 3: Variation in LL & PL with increasing percentage of silica fume

Table No. 4.: Test results of Atterberg's limits of pure soil and silica fume treated soil

Samples	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Pure Soil	45.81	36.63	25.05
95% ES + 5% SF	43.83	36.96	26.86
90% ES + 10% SF	43.73	34.65	25.90
85% ES + 15% SF	47.40	36.01	29.05

3.1.4 SOIL CLASSIFICATION

Soil classification according to USCS classification system (ASTM D2487) and AASHTO classification system (ASTM D3282) required particle size analysis and Atterberg limits results, after performing both test untreated soil or black cotton soil was classified as **CH- fat clay** (USCS) and **A-7-6** (AASHTO).

When black cotton soil was partially replaced with different percentages of silica fume soil classification remains the same as CH and A-7-6 soil.

When the soil was partially replaced with different lime percentages soil classification changes from CH to ML (Silt) and in case of AASHTO classification system it changes from A-7-6 to A-7-5.

When the soil was partially replaced with mixture of lime and silica fume at different percentages soil classification changes from CH to MH (elastic silt) than to ML (Silt) and in case of AASHTO classification system it remains same throughout A-7-6.

3.1.5 UNCONFINED COMPRESSIVE STRENGTH OF UNTREATED SOIL AND TREATED SOIL

To check the unconfined compressive strength (UCS) of the untreated soil, sample was prepared and tested according to ASTM D2216. 152 kpa is the calculated strength for the untreated black cotton soil.

The variation in unconfined compressive strength of soil treated with silica fume at different dosages of 5%, 10%, and 15%. When silica fume was partially replaced in black cotton expansive soil, unconfined compressive strength was increased up to the value of 160 kpa at 5% replacement and then decrease up to 15% at the value of 156 (less value than the pure soil). Table 4.15 shows the test results of unconfined compressive strength with increasing percentages of additives.

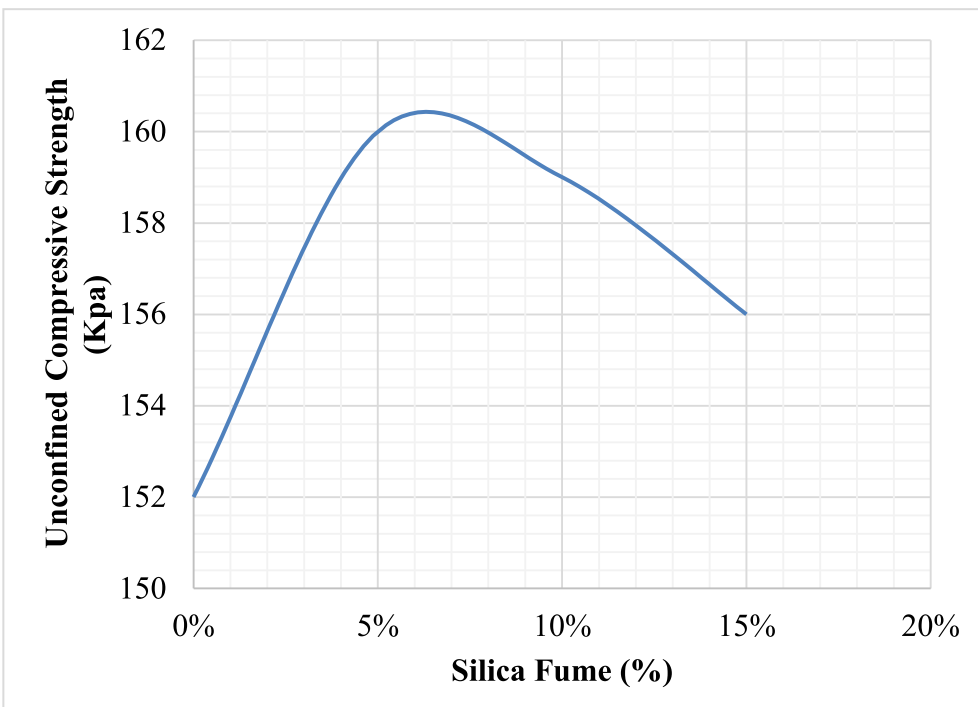

Figure 4.: Variation of unconfined compressive strength with increasing percentages of silica fume

Table No 5: Test results of unconfined compressive strength of pure soil and treated soil with additives percentages

3.1.6 SWELL TEST

Expansive soils have property to swell under different conditions and in order to control such properties different stabilizer were used and their results were analyzed , silica fume shows positives results in reducing the swell potential of the expansive soil (Nandipur region soil). When pure soil was subjected to swell test in accordance with ASTM D4546, testing for checking the swell potential of the soil, calculations shows a value of 6.3% swell.

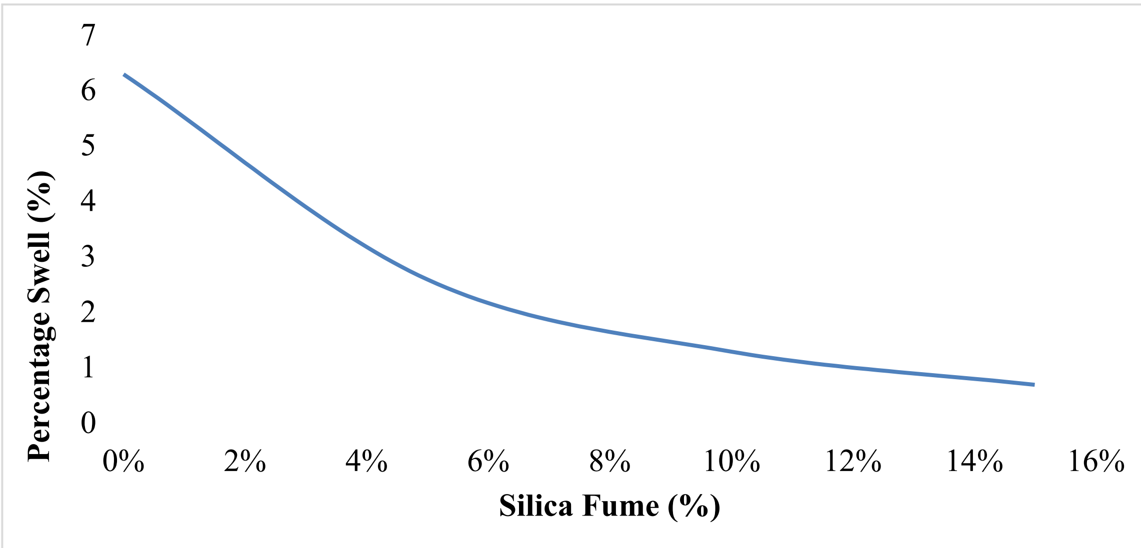

Figure .5: *Variation in swell percentage with increasing percentage of silica fume*

When soil was partially replaced with silica fume, there is a drop in the value of swell percentage was calculated. Figure 4.32 shows a variation in swell percentage with increasing percentage of silica fume, it shows decrease from 6.3% to 0.7%. Table 4.6 showing the test results of all the additives being partially replaced in black cotton soil to calculate the swell potential of the soil.

Table .6: Test results of swell potential of untreated and treated soil with increasing percentages of additives

Swell Percentage (%)	
Additives Percentage	**Silica Fume**
0%	6.3
5%	2.6
10%	1.3
15%	0.7

3.1.7 MOISTURE-DENSITY RELATIONSHIP

A compaction test was carried out using modified effort. The results of this test showed the maximum dry density (MDD) of 119.5 lb/ft3 and optimum moisture content (OMC) of 14.00 percent. ASTM D1557-12 was followed to perform compaction tests

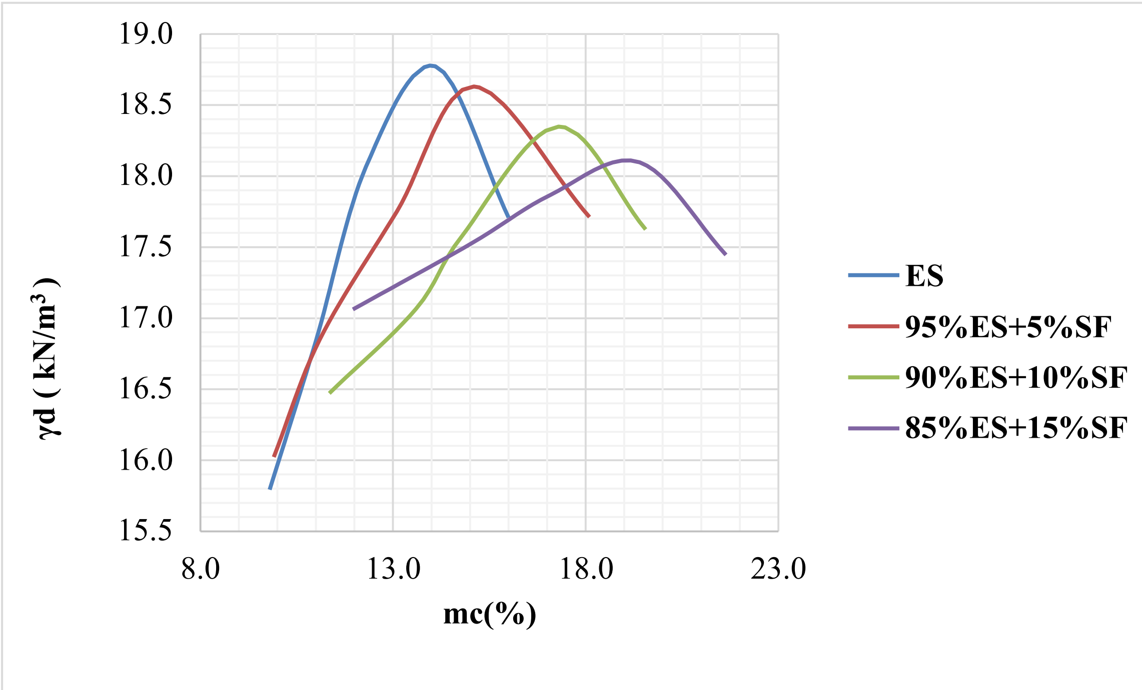

Figure .6: Relationship between dry density and moisture content of silica fume mixed soil and pure soil

Figure 4.18 shows a moisture density relationship curves with increasing percentage of silica fume and test results are shown in table 4.9, It was calculated from the results of moisture density relationship after partial replacement of additives that there is constant decrease in the value of MDD and increase in the value of OMC, when silica fume was replaced there is drop in the value of MDD i.e., from 119.5 lb/ft3 to 114.2 lb/ft3 and increase in the value of OMC from 14.00% to 18.4%. Table 4.10 shows the maximum dry density and optimum moisture content values for each percentage of silica fume being added to sample.

Table .7: MDD values of pure soil and silica fume treated soil

Samples	Maximum Dry Density (%)	Optimum Moisture Content (%)
Pure Soil	119.5	14.0
95% ES + 5% SF	118.0	14.4
90% ES + 10% SF	116.8	15.7
85% ES + 15% SF	115.1	17.1

3.1.8 SPECIFIC GRAVITY

Specific gravity determination, of a particular type of soil has a main role in estimation of its void ratio. Value of Gs is also linked with shrinkage parameters and hydrometer analysis closely. Therefore, specific gravity was measured carefully according to ASTM standard 854-10 (in 100 ml pycnometer with vacuum pump).

The specific gravities of natural clayey soil, 10%, and 20% of SF.

Table .8: Specific gravities of samples used

	Natural soil	5% SF	10% SF	15% SF
Gs	2.4	2.5	2.5	2.5

3.1.9 COMPACTION CHARACTERISTICS

Modified Proctor tests were carried out on natural soil and on its mixtures with SF in different proportions. The results of the compaction tests performed to assess the optimum water contents and maximum dry densities are given in Figure 4.6. A summary of compaction test results are given in Table 4.3. As can be observed additives caused optimum water content to increase while decreasing the maximum dry density in general

Table .9: Compaction test results

Sample	Optimum water content (%)	Maximum dry density (g/cm3)
8% SF	29	1.63
10% SF	24	1.60
12% SF	25	1.65
14% SF	26	1.64
16%SF	30	1.67

4. REFERENCES

1) Rao, K., Pranav, D., and Anusha, M. (2012), "Stabilization of Expansive Soil with Rice Husk Ash, Lime and Gypsum –an experimental study", *International Journal of Engineering Science and Technology* Vol.3 No.11 pp, 8076-8085.
2) Sonebi, M. (2010), "Optimization of Cement Grouts Containing Silica Fume and Viscosity Modifying Admixture", *Journal of materials in civil engineering*, pp.332-342.
3) ROY, T., Chattopadhyay, B., and Roy, S. (2009), "Effect of Lime on the properties of subgrades of roads with addition of Rice husk ash", IGC, Guntur, India, pp. 175-177.
4) Khandaker, M., and Hossain, A. (2011), "Stabilized Soils Incorporating Combinations of Rice Husk Ash and Cement Kiln Dust", *American Society of Civil Engineers*, pp. 1320-1327.
5) Practical Action Technology Challenging Poverty. (2009). Pozzolanas - Rice Husk Ash (RHA) and Pulverised Fuel Ash (PFA) - Technical Brief. Retrieved from:http://practicalaction.org/pozzolanas-rice-husk-ash-and-pulverised-fuel-ash.
6) Pham, V. "Utilization of Rice husk ash in Geo-Technology, Applicability and effect of the burning conditions", Ms Dissertation, Faculty of Civil Engineering and Geosciences, Delft University of Technology, July 2012.
7) Ipek, T. (1998). Stabilization of Expansive Soils Using Lime, Cement and Fly Ash (Doctoral dissertation, MS Thesis).
8) Basha, E., Hashim, R., Mahmud, H., and Muntohar, A. (2005),"Stabilization of residual soil with rice husk ash and cement"

11th International Civil Engineering Conference (ICEC-2020)
"Integrating innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

FOUNDATION ON SWELLING SOILS

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Abstract

This study is based on an insight & experience of working in swelling soil areas. It is a guide on how to identify swelling soils in field & laboratory with ways & means to reduce or prevent upward heave of foundation due to swelling. The study discusses pros & cons of different soil stabilization, drainage control techniques & preventive measures.

Three Case studies of existing structures (cracked due to heave) are analysed by laboratory test results. Chens & Constant Volume Tests were performed to evaluate % Free Swell & Swell Pressure. Appropriate choice of foundation is recommended for future new structures in those areas. Remedial measures are also discussed for existing structures laid on swelling soil which are experiencing upward movement & subsequent cracking. This study is a guide for Geotechnical & Structural Engineers to combat swelling pressure by right choice of foundation & adopting preventive measures.

Keywords

Liquid Limit (LL), Plasticity Index (PI), Free Swell (FS), Swelling Pressure (SP)

1) INTRODUCTION

Swelling soils are those which swell considerably upon increase in moisture (even while under load) & shrink on its removal. Swelling soils are clayey soils (sands & gravels do not swell) & contain highly plastic clay minerals like Montmorilonite, Illite & Kaolite.

Numerous structures constructed on swelling soil have experienced significant damage due to differential heave. Differential movements redistribute loads of the structure on the elements of foundation & cause large changes in moments & shears not accounted for in design.

Structures most often damaged due to swelling soils include Foundation & Walls of Light Loaded Buildings, Highways, Canals, Reservoir Linings & Retaining Walls.

Expansive or Swelling soils were recognized in 1930. Prior to this all soil problems were attributed to settlement only. Today there is a worldwide interest in Expansive Clays & Shales & many conferences have been held as losses caused by swelling soils are similar to a catastrophic damage.

2) HOW TO RECOGNISE SWELLING SOILS

2.1) Field Identification:

In the field swelling soil are present where the topography in uneven. They are treacherous in nature as visual inspection shows the soil is very stiff / sticky & the chances of lightly loaded structures cracking due to soil problem are rather remote. Another unique feature of swelling soil is presence of trees & mesquites in the vicinity showing subsurface soil with a high affinity for moisture which is characteristic of swelling.

2.2) Laboratory Identification:

The capacity to swell depends on the amount of clay minerals – Montmorillonite, Illite & Kaolite present which can be identified by tests such as

X-Ray Diffraction

• Electron Microscope etc. etc

However such tests are expensive & uneconomical for practical engineers.

2.2.1) Grain Size Analysis:

Since swelling is only associated with clays, particles passing # 200 sieve are always very high & mostly above 80%. If % passing is less it indicates presence of sand & as such degree of swelling in such soils in medium to low. It is also noted that the amount of colloidal content in swelling soils is more than 30%. (colloidal content is particle size less than 0.002 mm)

2.2.2) Atterberg Limit:

Plastic Index (PI) & Liquid Limits (LL) are a reliable Index Tests to recognize swelling soils & their potential to swell. Following can be a good reference.

PLASTIC INDEX	DEGREE OF EXPANSION
00 – 15	No Swelling / Low
20 – 30	Medium
35 – 40	High
PI > 45	Very High

Liquid Limit is a very reliable & instant method to recognize presence of the swelling clays. Mostly the LL value of 50 or above indicates swelling potential. As per Unified Soil Classification System (USCS) swelling soils are classified as CH / OH.

3) EVALUATING SWELLING POTENTIAL:

Swelling Potential is measured using oedometer used in most soil laboratories for measuring consolidation characteristics.

Different authors have recommended different procedures to evaluate % free swell & swell pressure. The most reliable methods among those are:

- Chens Method

3.1) Chens Method:

For this method the specimens is placed in an oedometer with the seating load & initial dial gauge reading is noted. The sample is than saturated (flooded). A swelling clay will start expanding as noted on the dial gauge. Final dial gauge reading is noted after 48 hours. Now load is applied until the initial reading is retrieved. The load which retraces initial reading is the swelling pressure.

The Free Swell can be calculated from:

$$\% \text{ Free Swell} = \frac{h_2 - h_1}{h_1} \times 100 \qquad \text{(A)}$$

h_1 – Initial sample height h_2 – Final sample height

Note: Each load to be maintained for 24 hours.

3.2) Constant Volume Test:

This is a reliable method but needs constant vigilance. Sample is set in the ring & initial dial gauge reading noted. The sample is them flooded to saturate & absorbs moisture.

As soon as the soil starts swelling & dial gauge reading moves, loading is applied & sample not allowed to heave. The final load at which no more expansion is observed is swelling pressure. % swell is calculated similar to Chens Method (equation A).

4) REMEDIAL MEASURES

Remedial measures can be divided into following:

i. Modified Foundation Design
ii. Soil Stabilization
iii. Preventive Measures
iv. Drainage Control Technique

The first two are for proposed new structures whereas drainage control & preventive measures are applicable for both new & existing structures. New structures can be prevented from swelling hazard but for existing structures a complete solution is not yet found.

4.1) Modified Foundation Design

4.1.1) Individual Footings

Individual Footings are designed where the thickness of swelling stratum is thin. Footing pressure penetrates upto shallow depth & negates heave for a shallow layer. The size of footing is proportioned (decreased) such that it imposes pressure above swelling pressure but certainly not above Allowable Bearing Capacity. The other option to increase foundation pressure is to increase the spans (distance between columns). To minimize differential heave individual footings are tied by well-designed tie beams. Such beams are kept 10 – 15 cms above ground & the gap between ground & tie beam is Back-Filled by granular soil or collapsible cardboard. Depth of placement for individual footing can also be increased to help resist heave by increased Back-Fill load.

4.1.2) Slab-on-Ground (Raft)

This may be sometimes costly but is a reliable method to resist heave. Many engineers consider Raft as dangerous but I believe Raft is safe & reliable to combat swelling - unless the groundwater is shallow & just beneath slab. The slab design must incorporate negative moments due to upward pressure. Mostly the slab design is governed by negative movements.
Raft can also be perforated with gaps. This perforation decreases the contact area & subsequently increases imposed pressure to combat upward heave.

4.1.3) Slab on Grid Beam (Raised Floor)

This is an effective way to combat upward heave for light loaded structures. The raised floor consists of a 10–15 cm thick Slab stiffened with underlying cross beams. Over these beams slab is cast 10 cm above ground. This method has construction difficulties like removal of shuttering. Gap between soil & slabs can be filled with granular soil or cardboard to support fresh concrete without use of shuttering.

4.1.4) Piled Foundation:

A proper choice of length & diameter of piles can result in an economical & safe solution. Piles can be short, seated just below a non-swelling stratum or below groundwater level. Such piles are floating piles where only skin friction resist uplift.

4.2) Stabilization Technique

The Swelling Potential of soil may be reduced by soil stabilization. The methods include:

i. Flooding or Pre-wetting
ii. Remolding the Soil
iii. Soil Replacement
iv. Mixing Chemicals

4.2.1) Flooding or Pre-wetting:

In this method the building area is flooded & soil allowed to swell prior to construction. This method has many assumptions & is not much reliable.

4.2.2) Remolding the Soil:

Excavating swelling clay & then Back Filling the same by compaction changes the natural fabric of the soil & thus its characteristic. Such remolding reduces swelling potential. If medium to coarse sand is mixed (sprinkled) with excavated clay, substantial drop in swelling pressure is achieved.

4.2.3) Soil Replacement:

A simple & easy solution for footings & slabs placed on expansive soils is to replace swelling soil with granular material (SP / SC). However this is only possible if the swelling clay stratum is shallow with thin stratum.

4.2.4) Chemical Stabilization:
Chemical stabilization of swelling clays is usually less effective compared to other techniques. The use of chemicals with swelling soils may show tremendous effect in laboratory experiments but in field the efficiency is low. This is because the swelling clay is very stiff with low permeability & as such proper mixing with chemicals is difficult.

- **Lime Stabilization**

Lime mixed with swelling clay reduces its plasticity & potential to swell. Lime stabilization was widely used in the past but now engineers prefer cement stabilization over lime.
Use of lime is only limited to highways & for building foundations it is seldom reported. Gain in strength due to mixing of lime takes months. Although use of lime for subgrade stabilization is questionable, it is popular due to economy.

- **Cement Stabilization (Soil-Cement Mixture)**

Cementing action of Portland Cement with clay is similar to lime mix as Portland Cement releases a large amount of lime. In addition Portland Cement increase the strength in a short span. The action of cement on clay reduces its PI & LL & thus potential to swell. Cement stabilization is recommended for under slab as it not only reduces swelling, but also produces a firm semi rigid base. If the sub-soil tends to swell the soil-cement mixture tends to distribute pressure uniformly, reducing damage caused by differential heave.

- **Calcium Chloride Treatment ($CaCl_2$)**

Calcium Chloride treatment reduces liquid limit & thus reduces the soils potential to swell. Calcium chloride solution is flooded on the area. However due to low permeability of clay percolation takes time to be effective.

4.3) Other Preventive Measures

4.3.1) Unground Tanks / Conduits
In an area of swelling clay the excavation for underground tank should be over size in plan. Extra 30 cm excavation, to be filled by sand. This process requires shuttering on soil side as well. Similarly for underground conduits extra excavation is required & filled by sand.

4.3.2) Retaining Walls
Swelling soils are very stiff & excavated cuts stand vertical without support. The structural engineer anticipates low active earth pressure. It should be understood that the swelling pressure is the uniformly distributed active pressure on the entire retained area. Negating or underestimating swell-pressure can result in overturning of the retaining wall.

4.3.3) Boundary Wall
Boundary wall in swelling clay areas to be supported by columns tied to tie beams. The tie beams should be cast 10 – 15 cm above ground & gap between tie beams & ground to be Back Filled by granular sand. Weight of block masonry on tie beams prevents the column from heave.

4.3.4) Existing Structure
There is no remedy for existing structures except that the rate of cracking may be reduced. Following precautionary measures may be adopted:

- No watering of lawn. Horticulture in pots.
- Construct pavement along periphery.
- Slope ground away from structure to prevent standing water seepage.
- Control on underground tank & conduit leakage.

4.4) Drainage Control Techniques

Swelling soils expand on increase in moisture. As such attempts have seen made to isolate water from foundations. Free water may be removed to prevent seepage by providing surface drains. However it is very difficult to stop moisture migration from an open (to sky) area to a covered area under a structure. This is because water has tendency to migrate from a warmer zone (open to sky) to a cooler zone which is present under a building shade. This results in increase of moisture & subsequent swelling.

4.4.1) Moisture Barriers

Horizontal & Vertical barriers can be installed around a building in the form of pavements or membranes. Sloping the ground away from the structure also holds & prevents accumulation of surface water. The purpose of barriers is to prevent seepage. It has been found that heave of soil is seldom observed around a petrol pump as it is covered by pavements.
Vertical apron can also be provided to prevent moisture migration towards a structure. However concrete aprons need constant care & vigilance as the apron itself can heave & allow moisture.

4.4.2) Peripheral Drains

Peripheral Drains can be installed around a structure where water can flow by gravity to minimize general wetting of foundation soil. The depth of drains should be atlases 45–60 cm below floor level to be effective.

5) CASE STUDIES

In Karachi Area following portions are underlain by Swelling Soils.

	Approx. Swelling Pressure
DHA Phase II around Abu Bakar Masjid	1.3 – 1.8 (kg/cm^2)
DHA Phase V around Saudi Consulate	1.2 – 1.6 (kg/cm^2)
Site Industrial Area	1.4 – 2.3 (kg/cm^2)
Landhi Industrial Area	0.8 – 1.2 (kg/cm^2)
Hill Park Area	1.4 – 2.1 (kg/cm^2)
Rohail Khand Society	1.6 – 2.5 (kg/cm^2)
Gulistan-e-Jauhar Block-III	1.1 – 1.4 (kg/cm^2)

The cause of damage of a structure can be ascertained, weather the cracking is due to swelling (heave) or settlement. For swelling following can be observed.
The pattern of crack is such that the external wall leans outward resulting in vertical cracks (mostly) with some diagonal cracks. Horizontal cracks are seldom. Large cracks are always observed at top where the slab tries to restrain wall movement & cracks appear. The doors & windows are misaligned & jammed. floors heaves & clear cracks are observed in tiled areas, such as bathrooms / kitchen etc. Pavements & Roads adjacent to the structures are found cracked. With such indicators & if the area topography is also uneven one can certainly attribute the damage to swelling & not settlement.
Following are the case studies which have seen thoroughly analyzed & discussed here.

5.1) Mehran Engineering University (Jamshoro Sindh):

Sir Syed hostel was studied. It has totally cracked with 08 – 10 cms wide cracks. It was constructed in 1960. Cracks started to appear 6 months after construction when there were heavy rains. Most cracks are Diagonal & Vertical. The floors have heaved 10 – 12 cms. Severe moments were noticed with doors & window misaligned. Test Pits were excavated & following was the summary of laboratory tests results.

% Passing # 200 Sieve	=	82%
Liquid Limit	=	63%
Plastic Index	=	46%

Method	Swell Pressure (kg/cm^2)	% Swell
Chens Method	2.35	11.0
Constant Volume Method	2.45	5.75

- **Observation / Recommendation**

As observed the area has uneven topography. Sub-soil has high LL & cracks pattern show heave due to swelling. The hostel was laid on Individual Footing at 1.2 meter depth. It is now recommended to lay foundation on Raft (Slab on ground) or short Piles for future constructions.
There is no remedy for the existing structure & it has to be demolished.

5.2) Earth Satellite Station (Dehmandro Sindh):

Situated 40km from Karachi the earth satellite station has 3 structures - Machine Room, Dormitory & Platform for Antenna. Both Dormitory & Machine Room were found totally cracked. The machine room was post tensioned by heavy jack to keep it intact. Dormitory was supported on individual footings & developed 08-10 cm wide cracks. The floor has heaved by 10-12 cm. Doors & windows are misaligned / jammed. Underground conduits are leaked, with block masonry & tiles severely cracked. The whole structure has been left redundant.
Two Test Pits were excavated & samples extracted. It took 8 hours for 2 laborers to dig 1 pit 8 feet deep indicating the very stiff nature of soil.
Following is the summary of Laboratory Results:

% Passing # 200 Sieve = 74%
Liquid Limit = 50%
Plastic Index = 29%

Method	Swell Pressure (kg/cm^2)	% Swell
Chens Method	1.20	3.82
Constant Volume Method	1.40	3.77

(Low Swell pressure due to 20-25% sand)

- **Observation / Recommendation**

From uneven topography of the area, visual inspection of crack pattern, jammed doors / windows, raised floor & laboratory result it is evident that the damage at Earth Satellite Stations Dehmandro is due to heaving of foundations & not settlement.
The existing structures have no remedy. Future structures should be supported by slab-on-grid or short piles. Slab on Ground may not be suitable as groundwater in this area is shallow.

5.3) Defence Housing Authority Phase V (Karachi, Sindh):

In this area numerous Bungalows (more than 70) have sustained significant cosmetic damage due to cracks developed by heaving. More cracks appears after rainy season. In this study Bungalow # 30 on 9th Street has been investigated. This area is around Saudi Consulate. Cracks started to develop during construction & the contractor was blamed for shabby construction. By visual inspection it is evident that the cracks are due to swelling & subsequent heave of foundation. One Test Pit was excavated. Following are the laboratory results:

% Passing # 200 Sieve = 90%
Liquid Limit = 52%
Plastic Index = 17%

Method	Swell Pressure (kg/cm^2)	% Swell
Chens Method	1.20	4.17
Constant Volume Method	1.50	3.40

- **Observation / Recommendation**

The area has uneven topography. The boundary wall & roads are cracked. Severe cracks are observed in tiled bathrooms & kitchens. New cracks continue to appear & old cracks widen particularly after rains. It is thus evident that the damage to Bungalows is due to swelling. There is no remedy for existing structures except that the rate of cracking may be minimized by taking preventive measures such as stop watering lawn, sloping ground away from structure & repairing leaking severs. For new structure – soil investigation should be performed. After determining the swell pressure, groundwater table & loads of the structure modified footing should be designed. The best & safe way is to lay the light loaded structure on raft foundation which may be perforated. Retaining walls (if any) should be designed by considering active earth pressure equivalent to swell pressure.

6) CONCLUSION & RECOMMENDATION:

- In an area of uneven topography presence of swelling soil is probable.
- A soil with LL $\geq$ 50 indicates presence of swelling potential.
- Chens method of evaluating swell potential is most reliable & easy to perform.
- To negate swelling pressure Individual Footing sizes may be decreased or column spacing increased.
- Raft Foundation is a safe choice to combat heave of foundation unless groundwater is shallow.
- A perforated Raft can be designed to increase imposed pressure.
- Swelling Potential of soil can be decreased by excavating & Back Filling the same remoulded clay after mixing (sprinkling) coarse sand.
- Except Soil-Cement mixture chemical stabilization is not effective for building foundation.
- Retaining Wall must be designed considering Swell Pressure as Active Earth Pressure.

In an area of uneven topography, sub-soil investigation must be performed even for light loaded structures with an object to precisely evaluate the depth & thickness of swelling soil stratum, groundwater table, swelling pressure etc. The geotechnical consultant must mention Minimum & Maximum stresses to be imposed to combat heave. The decision for choice of foundation should be mutually decided by both Structural & Geotechnical Engineers.

REFERENCES

- Foundation on Swelling Soils *“F.H Chen”*
- Geotechnical Engineering *“Donald P. Coduto”*
- Foundation Engineering *“Pech Hansen Thornburn”*
- Soil Mechanics *“Alam Singh*

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Evaluation of Fatigue Characteristics of Reclaimed Asphalt Pavement Binder

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Abstract

Pakistan is currently passing through a crucial situation, trying hard to cope with infrastructure development and economic crisis. There are many highways and motorways which needs rehabilitation and maintenance after every few years. They are being replaced with newer layer, wasting the old Asphalt Pavement. The future of our generations and their sustainability largely depends upon natural resources, because it is the natural resources that are used throughout every process of construction and development going on in modern era. To reutilize the material of aged asphalt pavement instead of wasting it, recycling is one of the best options. The main objective of this research work was to evaluate the fatigue characteristics of RAP binder. Samples of Reclaimed Asphalt Pavement is taken from Motorway M-2. Binder is recovered using binder recovery apparatus. To evaluate recovered binder's fatigue characteristics, detailed testing was carried out in laboratory on reclaimed asphalt pavement binder and when RAP binder modified with the virgin binder in best optimum percentage. The whole testing process constituted on two phases. In first phase, Basic conventional testing as well as advance tests were performed on binder sample extracted from Reclaimed Asphalt Pavement. In second phase of testing, whole testing mechanism was performed on Reclaimed Asphalt Pavement Binder added with optimum amount of virgin binder. Same was repeated for the sample of virgin binder. Basic testing involved "Penetration, Softening Point, Flash & Fire Point, Ductility, Solubility and Specific Gravity tests while advance testing was done focusing on Dynamic Shear Rheometer, PG-Grading and Frequency Sweep Test. Results depicted that the fatigue properties of RAP binder were enhanced after modification with (softer grade of) virgin binder.

Keywords

RAP binder, Fatigue Properties, Economic Crisis, Sustainability, Natural Resources,

1. Introduction

Transportation is the backbone of a country's economics. Throughout the whole year, enormous financial funds are being spent on maintenance and rehabilitation of previously constructed roads and for new roads projects. In Pakistan, every year in yearly budget, billions of rupees are allocated only to serve this purpose. Pavements play an important role in prosperity and development of a country. And with increase in its importance, its usage is enhancing day by day. As a result, pavements are disintegrating and their deteriorations process initiate with the passage of time. So, there occurs need for their maintenance and rehabilitation. There are many distresses and factors due to which failure of pavement occurs. The main factor which is responsible for deterioration of pavement is phenomenon of Fatigue. Fatigue cracking is one of major types of distresses like fatigue cracking, rutting and low temperature cracking of flexible pavements. Possible problems which occur because of Fatigue Cracking are that cracks allow moisture penetration, it is sign of structural failure, roughness which might further worsen to a pothole. Probable causes which make Fatigue Cracking happen are decline in pavement load supporting features, loss of subbase, base or subgrade support, stripping on the bottom of the hot mix asphalt layer. Because of above mentioned distresses, thousands of kilometers pavements get deteriorate and their functional as well as structural performance become too low that these need maintenance and rehabilitation to match up its structural integrity and functional performance. This cause huge expenditures as well as numerous usage of natural resources like aggregates, binder and other materials that are used in construction of pavements.

1.1 Research methodology

Reclaimed asphalt pavement sample was taken from Lahore Islamabad Motorway M-2. Centrifuge apparatus and binder recovery apparatus were used to extract the bitumen content from reclaimed asphalt pavement. After extraction of bitumen from RAP sample, detailed testing was done to evaluate the fatigue characteristics of RAP binder. Virgin binder which was used had 80/100 ARL pen grade. Three samples were prepared i.e. RAP binder, Virgin binder and RAP binder in addition with optimum amount of Virgin binder. Optimum amount of virgin binder was obtained through preparation of job mix formula (JMF) and comparing the amount of binder obtained through extraction from reclaimed asphalt pavement. Here is the summary of research Methodology in Figure 1,

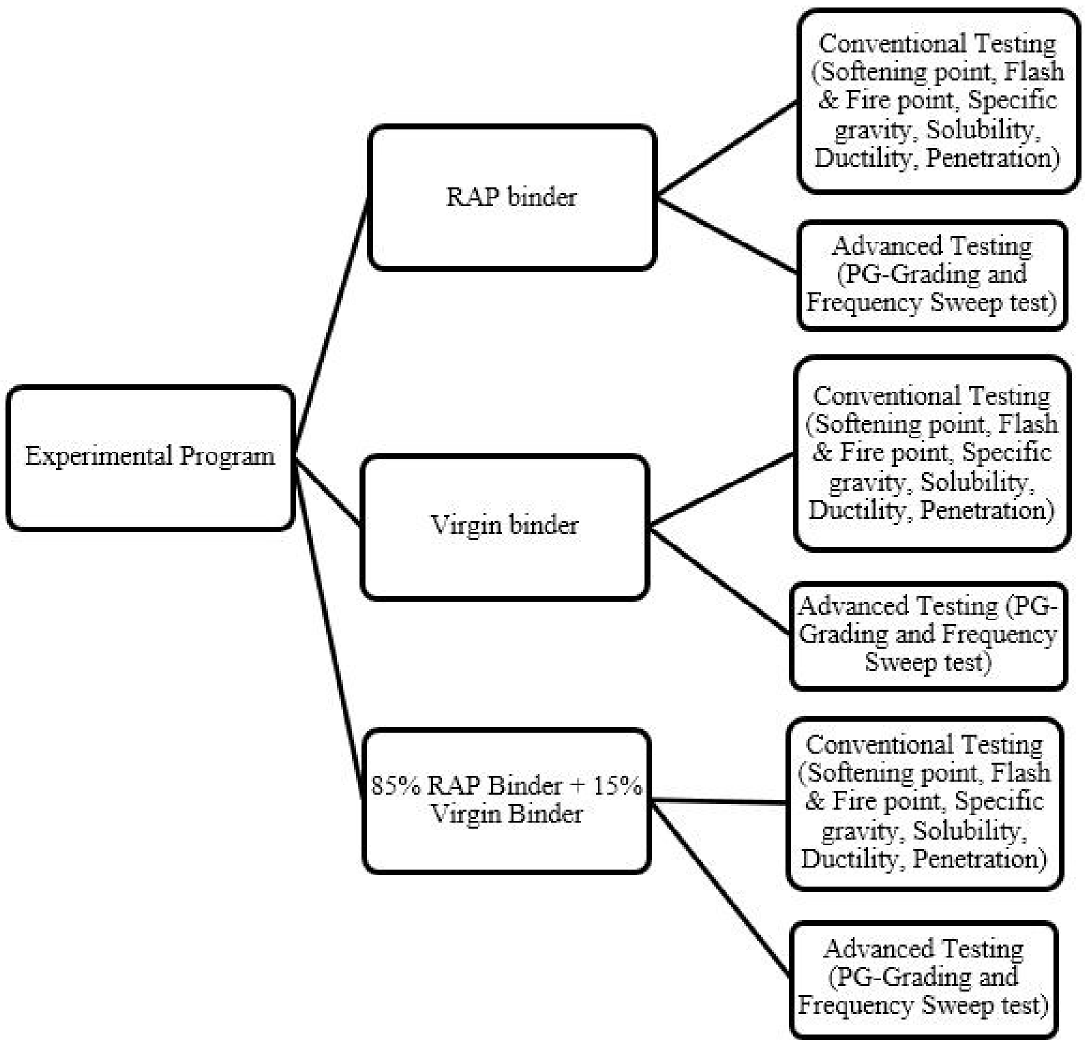


Figure 1: Research Methodology

1.2 Literature Review

Throughout the Transportation Industry, the use of RAP is increasing due to its positive environmental effects and economic advantages as compared to the use of virgin aggregate and binder. There is a limited amount of natural resources throughout the whole world, making reclaimed asphalt pavement, the most economical choice for contractors and government agencies. Reclaimed asphalt pavement also created possible opportunity to replace some of the binder that would be used already in the mix, rendering less need for usage of virgin binder. Due to the aging of the reclaimed asphalt pavement, it is also considerably stiffer, which leads to the enhanced strength, resistance to rutting and susceptibility to moisture (Al-Qadi, 2009) than the softer grade virgin binders. However, the stiffness might lead to some other problems likes of cracking, but there are many admixtures, rejuvenators that can be combined with the reclaimed asphalt pavement to eradicate this problem. Reclaimed asphalt pavement usage is growing day by day and this will continue to grow and lead to green sustainability and perpetual increase of roads system throughout the world.

Reclaimed asphalt pavement is the term that is being used to describe recycled or roto-milled asphalt paving materials containing considerable amount of asphalt and aggregates. Reclaimed asphalt pavement has been utilizing into pavements successfully for past many years and thus providing a huge amount of savings of financial and natural resources. Every year, approximately 90 million tons of reclaimed asphalt pavement are being reused, which is almost twice as much as the collective total of recycled paper, plastic, glass and aluminum and. When HMA pavements advance to the end of their operational service life, the material in them holds substantial value. Reclaimed asphalt pavement is a treasure trove of recycled road building material. The aggregate has already undergone the processing, permitting, blasting and washing. Thus, making reclaimed asphalt pavement usage economical.

The huge cost benefit originates from the recovery of the asphalt binder from reclaimed asphalt pavement. Besides the cost benefits, the use of RAP presents an environmentally beneficial method of recycling. Reusing of reclaimed asphalt pavement should follow these attentions such as recycling and reuse must offer economic, engineering and environmental benefits, determination of the use of recycled materials should include an initial review and feasibility report of engineering, economic and environmental stability, recycled materials should get first priority in material selection, an assessment of economic benefits should be performed during the selection process, limitations that prohibit the use of recycled materials without any technical and engineering basis should be excluded from specification.

1.2.1 Study of RAP Binder in Pakistan

Reclaimed asphalt pavement material was attained in the shape of chunks from two places (Mandra & Nowshera) sideways national highway N-5, Pakistan. No proper stock piling of the RAP materials was incorporated and they were subjected to severe aging and weathering which has adverse effects on the rheological properties. The virgin binder of 60/70 pen grade was obtained from Attock oil refinery limited (ARL), Rawalpindi. Attock oil refinery limited is consuming local weighty rudimentary oil mixture of 7%-10%, crude comprising 3%-5% asphaltenes for manufacturing of pen grade 60/70 and 80/100 asphalt. Maximum of local heavy crude oil are generated in northerly portion of the Pakistan.

In Pakistan, the binders are still graded by penetration value and no PG grade binders are available for the experimental work. In previously research this binder was graded as PG 58-22 with softening point of 66.5oC and flash point of 255oC. The ARL 60/70 binder was used in different percentages (0, 10, 20, 30, 60 and 100) with aged binder. For low RAP amount (5 to 20 percent) the effect of the properties of the reclaimed asphalt pavement material on the final products is too minimum so there is no need to perform the tests. On the other hand, for high reclaimed asphalt pavement amount, the reclaimed asphalt pavement properties would have substantial effect on the performance and quality of the final products, thus they require to be properly analyzed and tested. Using RAP not only lessens the cost of new asphalt blends but also preserves the natural resources.

The residual binders attained from two reclaimed asphalt pavement sources using solvent extraction and binder recovery methods were mixed with a virgin binder in different fractions. Penetration, ductility, stiffness, dynamic modulus and viscosity of the different blends were examined. Viscosity, penetration and PG grading blending charts were established created on the related test data. It was concluded that the properties of the blends rely on the individual properties of the binders. The stiffness of the binder is growing with enhancing reclaimed asphalt pavement binder. The Low temperatures also affect the m-value and cause the increased flexural creep stiffness of the binders and their blends. This increased stiffness tends to cause low temperature cracking in the pavements. The high RAP content and low temperature results in decrease creep rate. The decrease rate for these values was very small and this small rate of change suggest that increasing RAP content does not decrease the creep rate significantly. This research only addresses the binder study so to calculate reclaimed asphalt pavement in asphalt mix design along with the other factors like volumetric properties and aggregates require appropriate attentions. (Arshad Hussain and Qiu Yanjun 2013)

1.2.2 Summary

The use of reclaimed asphalt pavement in different road related applications become more widespread in last two decades. The materials present in old asphalt pavements have residual value even when the pavements themselves have reached the ends of their service lives. Recognizing the value of those existing asphalt resources, agencies and contractors in many countries have made wide use of RAP binder in producing new asphalt pavements for decades. Usage of reclaimed asphalt pavement has recognized to be financially, economically and environmentally positive. In addition to the above, the performance of pavements with properly prepared recycled asphalt in terms of fatigue, rutting, thermal resistance and durability proved to be satisfactory (Al-Qadi 2007).

RAP is typically used between 10% to 30% in hot recycling asphalt mixtures. The environmental and financial restrictions are forcing the researchers to incorporate high percentage of RAP in pavement construction. One of the main barriers in achieving this goal is the increased stiffness of the RAP binder. RAP mixes can have similar or better performance than virgin mixes if they are designed following the balanced mix design procedure. Cracking performance of asphalt mixes, different from rutting, is strongly connected with pavement structure. It is extremely difficult to propose a single cracking requirement for all applications. Cracking performance is influenced by many factors, such as traffic, climate, existing pavement conditions for asphalt overlays, and pavement structure and layer thickness. There is a terrible need to develop a balanced RAP mix design and performance evaluation system for project-specific service conditions, including traffic, climate, existing pavement conditions, etc. Overlays with mixes that contained 30 percent RAP performed as well as overlays with virgin mixes in terms of IRI, rutting, block cracking, and raveling. In terms of fatigue cracking and transverse cracking, virgin mixes edged the 30 percent RAP mixes. Thicker overlays improved pavement performance, except for rutting. Milling before rehabilitation decreased IRI, fatigue cracking, and transverse cracking but increased rutting. (R. West, J. Michael 2011)

1.3 Test data and trends

Conventional testing includes solubility test, softening point, penetration test, ductility test, specific gravity test and flash and fire point. All above mentioned conventional tests were performed in accordance with their ASTM standards. These tests were performed on all three specimens i.e. RAP binder, Virgin binder and RAP binder in addition with optimum amount of virgin binder ARL 80/100 pen grade. Tests results have been shown in Table 1 and basic testing results of Virgin binder 80/100 pen grade have been shown Table 2,

Table 1: Conventional testing outcomes of RAP binder

Sr. #	Standard	Test Name	Result	Remarks
1	ASTM D2042	Solubility	97.50%	Standard value is 99.9 %, our result is close to

	AASHTO T53	test		standard value.
2	ASTM D36 AASHTO T53	Softening point	71°C	Standard range for softening point is 40 to 80°C for virgin binder, our result is close to standard value.
3	ASTM D92 AASHTO T48	Flash and fire point	Flash pt. = 260°C Fire pt. = 268°C	Standard range for flash and fire point is 232°C to 400°C, our result is close to standard value.
4	ASTM D3142 AASHTO T166	Specific gravity	1.19	Standard range for specific gravity of virgin bitumen is 0.97 to 1.02, our specimen S.G is 1.19.
5	ASTM D5 AASHTO T49	Penetration test	46	Standard value of penetration at standard conditions (25°C, 100g, 5 sec) is 0 to 49mm. Our value is in between this range.
6	ASTM D113 AASHTO T51	Ductility	6.0 cm	The ductility value ranges from 5cm-100cm. Our value is within the range.

Table 2: Conventional testing outcomes of Virgin binder ARL 80/100 pen grade

Sr. #	Standard	Test Name	Result	Remarks
1	ASTM D2042 AASHTO T53	Solubility test	99.70%	Standard value is 99.9 %, our result is close to standard value.
2	ASTM D36 AASHTO T53	Softening point	47°C	Standard range for softening point is 40 to 80°C for virgin binder, our result is close to standard value.
3	ASTM D92 AASHTO T48	Flash and fire point	Flash pt. = 240°C Fire pt. = 248°C	Standard range for flash and fire point is 232°C to 400°C, our result is close to standard value.
4	ASTM D3142 AASHTO T166	Specific gravity	1.03	Standard range for specific gravity of virgin bitumen ranges from 0.97 to 1.02, our specimen S.G is 1.03.
5	ASTM D5 AASHTO T49	Penetration test	90	Standard value of penetration at standard conditions (25°C, 100g, 5 sec) is 0 to 49mm. Our value is in

				between this range.
6	ASTM D113 AASHTO T51	Ductility	Above 100cm	The ductility value lies between 5cm to over 100 cm. Our value is within the range.

All basic tests were done to characterize the properties of RAP binder. As RAP is an aged material, thus all results of basic testing were on higher sides except solubility, penetration and ductility which showed results on lower side as compared to the basic testing results of Virgin binder. All testing results showed greater values except solubility, penetration and ductility which showed results on lower side than mentioned in ASTM standards.

RAP binder was modified by adding optimum amount of Virgin binder of softer grade. Optimum amount of Virgin binder was determined by making JMF using binder extracted from RAP and adding different amounts of Virgin binder ARL 80/100 pen grade. Optimum amount which deduced was 15% of Virgin binder. 15% Virgin binder was added in 85% of RAP binder. Basic results of this specimen have been shown in Table 3,

Table 3: Basic testing outcomes of RAP binder 85% + Virgin binder 15%

Sr. #	Standard	Test Name	Result	Remarks
1	ASTM D2042 AASHTO T53	Solubility test	98.50%	Standard value is 99.9 %, our result is close to standard value.
2	ASTM D36 AASHTO T53	Softening point	60^{o}C	Standard range for softening point is 40 to 80^{o}C for virgin binder, our result is close to standard value.
3	ASTM D92 AASHTO T48	Flash and fire point	Flash pt. = 254^{o}C	Standard range for flash and fire point is 232^{o}C to 400^{o}C, our result is close to standard value.
			Fire pt. = 264^{o}C	
4	ASTM D3142 AASHTO T166	Specific gravity	1.13	Standard range for specific gravity for virgin bitumen varies from 0.97 to 1.02, our specimen S.G is 1.13.
5	ASTM D5 AASHTO T49	Penetration test	55	Standard value of penetration at standard conditions (25^{o}C, 100g, 5 sec) is 0 to 49mm. Our value is in between this range.
6	ASTM D113 AASHTO T51	Ductility	40cm	The ductility value ranges from 5cm-100cm. Our value is within the range.

85% RAP binder in addition with 15% of Virgin binder of 80/100 ARL pen grade showed basic testing results between basic testing results of RAP binder and Virgin binder. It showed less stiff behavior than RAP binder.

1.3.1 PG grading of specimens

PG-grading was performed to evaluate fatigue characteristics of reclaimed asphalt pavement binder. For each specimen, the complex shear modulus (G*) and phase angle (δ) at each specified frequency were determined. Parameters for PG grading were as;
Mean frequency: 10 rad/sec
Mean strain amplitude: 12%
Results of PG grading for RAP binder, 85% RAP binder in addition with 15% of Virgin binder ARL 80/100 pen grade and Virgin binder have been shown in Table 4, Table 5 and Table 6 respectively,

Table 4: PG grading outcomes of RAP binder

Material	Temperature ^{0}C	Mean Phase Angle ^{0}	G* (kPa)	G*.sin δ (kPa)	G*/sin δ (kPa)
RAP	58	61.7	76.7	66.52	85.97
	64	64.3	38.3	34.51	42.5
	70	67.7	19.9	18.41	21.5
	76	71.4	9.7	9.19	10.2
	82	74.9	4.64	4.48	4.81
	88	78.1	2.24	2.19	2.29

Table 5: PG grading outcomes of RAP binder 85% + Virgin binder 15%

Material	Temperature ^{0}C	Mean Phase Angle ^{0}	G* (kPa)	G*.sin δ (kPa)	G*/sin δ (kPa)
RAP binder + Virgin binder	58	60.4	54.2	47.12	62.3
	64	65.6	28.3	25.77	31.1
	70	69.6	13.6	12.75	14.5
	76	73.4	6.44	6.17	6.72
	82	76.9	3.04	2.96	3.12

Table 6: PG grading outcomes of Virgin binder

Material	Temperature ^{0}C	Mean Phase Angle ^{0}	G* (kPa)	G*.sin δ (kPa)	G*/sin δ (kPa)
Virgin binder	58	61.7	76.7	66.52	85.97

Specimen was found to fail at 58oC while mean frequency was 10 rad/sec and mean strain amplitude was 12%. Thus, Virgin binder can't perform well at higher temperature.

1.3.2 Frequency sweep test results of Specimens

For each specimen, phase angle (δ) and complex shear modulus (G*) at specified frequencies determined. These values are calculated by software programs from the measured values of shear load and shear displacement recorded as a function of time. This standard gives performance associated test procedure for the determining the complex shear modulus G*.

Frequency sweep test was performed on all three specimens i.e. RAP binder, Virgin binder and 85% RAP binder + 15% of Virgin binder ARL 80/100 pen grade. Frequency sweep test result for all three specimens i.e; RAP binder, Virgin binder and 85% RAP binder + 15% of Virgin binder ARL 80/100 pen grade have been shown in Appendix A, Appendix B and Appendix C respectively. Frequency sweep test showed that Complex Viscosity is decreasing with the increase in angular frequency and temperature imposes the same effect on Complex Viscosity as of angular frequency. Storage Modulus and Loss Modulus is increasing with the increase in angular frequency and with the increase in temperature, these two parameters increase. RAP Binder blended with Virgin Binder has greater Complex Viscosity than Virgin Binder when experimented at same temperature. Parameters for Frequency sweep test were as;

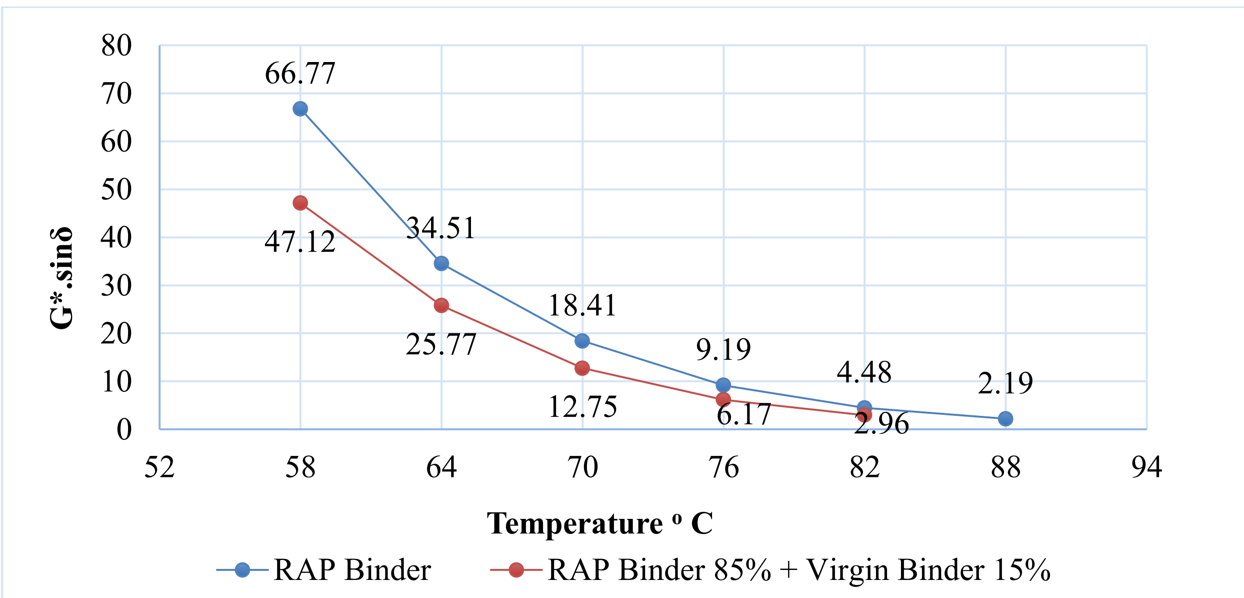
Temperature: 10oC-80oC

Frequency: 10-0.1 rad/sec

Strain: 10%

1.3.3 Tests Discussion

PG grading results for RAP binder and RAP binder 85% + Virgin binder 15% are shown and compared in Fig. 4.1, G*.sinδ is compared in mentioned figure for both specimens.

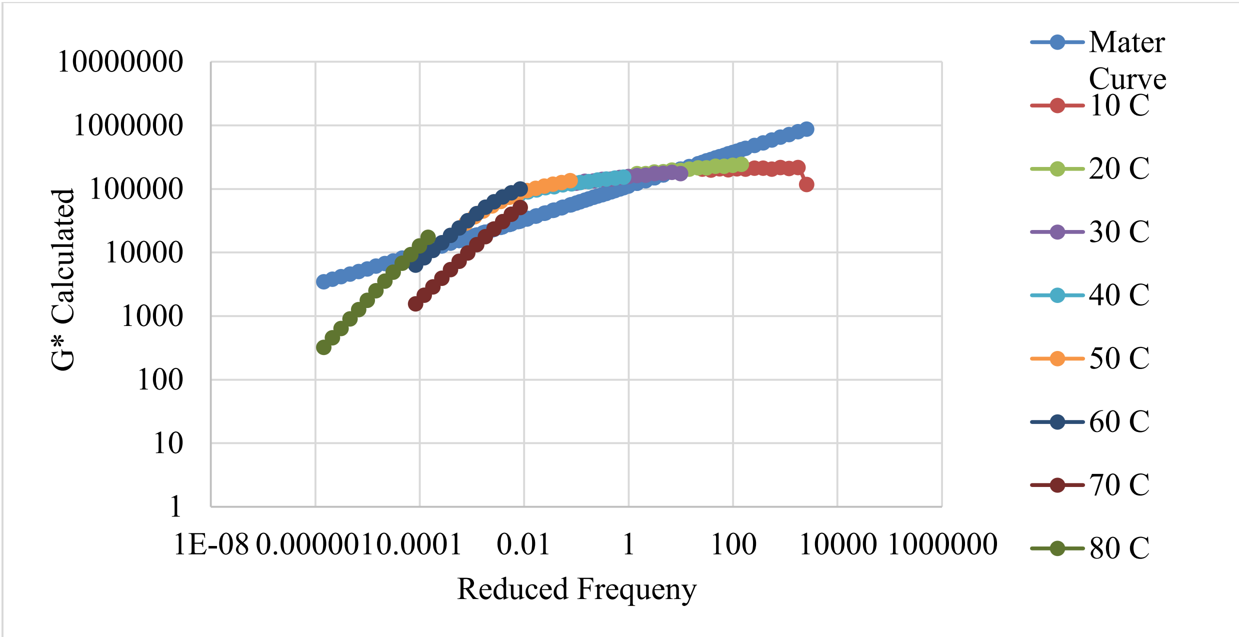


Graph 1: Comparison of Fatigue Parameter

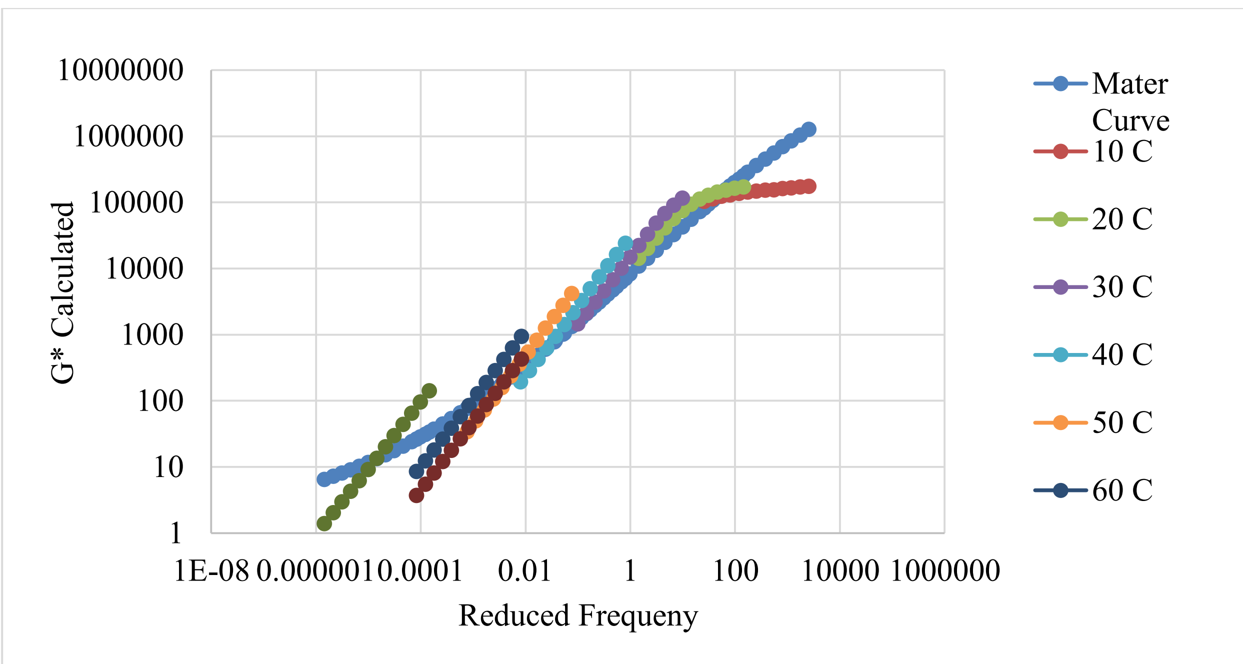
Comparison of results have shown that on lower temperatures, fatigue parameter for RAP binder shows greater value as well as on higher temperature also.

1.3.4 Master curves for specimens

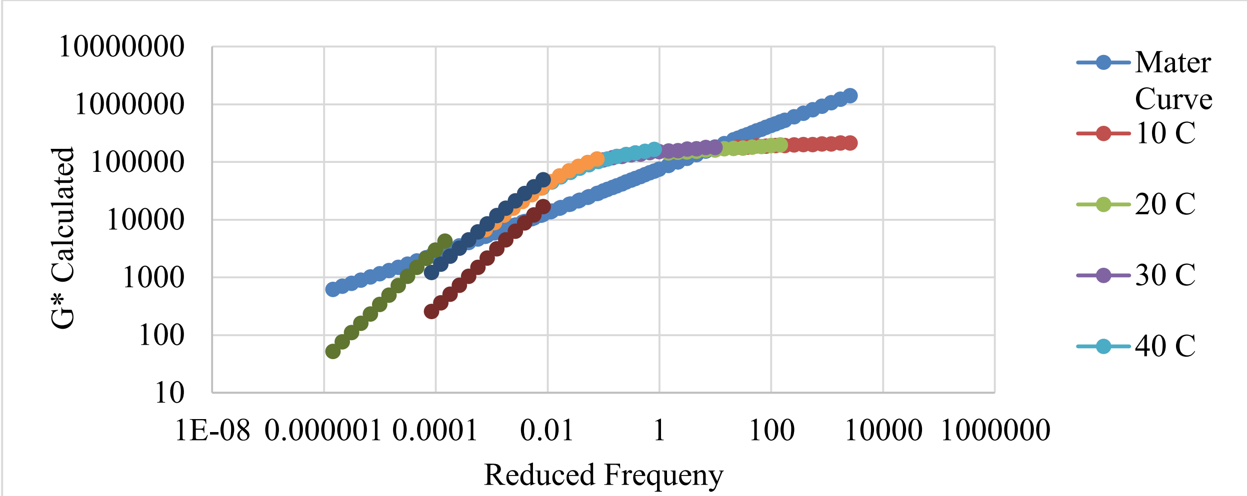
Master curve between G* and reduced frequency was developed for each specimen i.e. RAP binder, Virgin binder and RAP binder 85% + Virgin binder 15%. These master curves are given in Graph 2, 3 and 4. Master curves were plotted using sigmoidal model (Abedli Al-Haddad), (Booshehrian et. al),



Graph 2: Master curve between G* and Reduced Frequency for RAP binder



Graph 3: Master curve between G* and Reduced Frequency for Virgin binder



Graph 4: Master curve between G* and Reduced Frequency for RAP Binder 85% + Virgin Binder 15%

1.3.5 Result Analysis

Although fatigue parameter which obtained by performing PG-grading of only RAP binder shows greater values of G*.sinδ, this means that RAP binder shows more resistance to fatigue cracking and serves structural and functional performance better than when blended with optimum amount of virgin binder of softer grade. But this is not the only criteria of cracking. Because RAP binder is also more susceptible to temperature cracking. So, upon mixing of virgin binder of softer grade, fatigue characteristics were enhanced as compared to fatigue characteristics of RAP binder only. Frequency sweep test showed that Complex Viscosity is decreasing with the increase in angular frequency and temperature imposes the same effect on Complex Viscosity as of angular frequency. Loss Modulus and Storage Modulus is increasing with increase in angular frequency and with the increase in temperature, these two parameters increase. RAP Binder blended with Virgin Binder has greater Complex Viscosity than Virgin Binder when experimented at same temperature.

1.4 Conclusion

This research presents a lab based study that was arranged to assess the fatigue properties of reclaimed asphalt pavement binder by determining the conventional tests results, PG grading results and Frequency sweep results. The extensive evaluation and testing on the RAP binder, Virgin binder and RAP binder when blended with optimum amount (15%) of Virgin binder, using conventional and advanced testing, terminates on the following deductions.

- Greater percentage of RAP binder in specimen enhances the stiffness of specimen. But decreases when rise in temperature occurs. More amount of RAP binder increases the stiffness, viscosity and critical temperature of the blend, thus susceptible to temperature cracking at low temperature.
- It is endorsed that PG grade should be checked at beginning of every road and highway project as a parameter of quality control and quality assurance. Fatigue characteristics of the binder can be used to categorize sample and batch inconsistency and their consequent influence on the pavement functionality. PG grading must be preferred over pen-grade grading. PG grading of Virgin binder showed that specimen didn't pass even at 58oC but when PG grading RAP binder in addition with 15% of Virgin binder was done, the specimen didn't fail even after increasing temperature by 150% (88oC).
- RAP binder shows greater value of fatigue parameter as well as rut parameter in comparison of RAP binder + Virgin binder. Complex Viscosity is decreasing with the increase in angular frequency and temperature imposes the same effect on Complex Viscosity as of angular frequency. Loss Modulus

and Storage Modulus is increasing with the increase in angular frequency and with the increase in temperature, these two parameters increase. RAP Binder blended with Virgin Binder has greater Complex Viscosity than Virgin Binder when experimented at same temperature.
- Use of RAP binder with virgin binder of softer grade slows down the process of aging because RAP binder has already undergone the process of aging and oxidation of binder.

1.5 Recommendations

- More tests should be done to work out the optimum amount of virgin binder of softer grade that must be added to attain enhance fatigue characteristics of reclaimed asphalt pavement binder, as it is aged binder already.
- The objective of conducted research was to create a clearer thinking of using maximum amount of RAP binder along with the pavement performance properties such as fatigue characteristics and PG-grading. These experiments were meant to find feasibility of being able to use greater amount of RAP binder in the field, to create a truly sustainable pavement. To find this, RAP binder was used with virgin binder of softer grade. Some further tests need to be conducted to bring a better understanding of usage of greater amount of RAP binder.
- Rejuvenators and polymer modifiers can be utilize to reduce the amount of virgin binder required when used with RAP binder, as they would soften the aged binder more than just virgin binder.
- Further testing should occur with the addition of these rejuvenators, such as PG-grading, frequency sweep and fatigue resistance to find the true value of these rejuvenators.
- Once these tests are run, the cost benefit ratio can be compared to just using virgin binder to find the most economical way to use greater amount of RAP binder.

4. References

Arshad Hussain and Qiu Yanjun. "Effect of Reclaimed Asphalt Pavement on the Properties of Asphalt Binders". Procedia Engineering 54, 840 – 850. 2013

Al-Qadi IL, Elseifi M and Carpenter SH (2007), Reclaimed Asphalt Pavement- A Literature Review, Research Report Fhwa-Ict-07-001, Illinois Center for Transportation

Al Qadi I. L., Carpenter, S., et. al., "Investigation of Working Binder in Hot-Mix Asphalt Containing Recycled Asphalt Pavements", 88th Transportation Research Board Annual Meeting, pp 09-1262, Washington D.C., 2009

Al-Qadi, I. L., S. H. Carpenter, G.L. Roberts, H. Ozer, Q. Aurangzeb, Investigation of Working Binder in Hot-Mix Asphalt Containing Recycled Asphalt Pavements, Illinois Center of Transportation, Paper Number 09-1262, March 2009

Al-Qadi, I. L., S. H. Carpenter, Mostafa Elseifi, Reclaimed Asphalt Pavement- A Literature Review, Illinois Center for Transportation, Research Report FHWA-ICT-07-001, March 2007

Construction of a Complex Shear Modulus Master Curve for Iraqi Asphalt Binder using a Modified Sigmoidal Fitting DR. ABDULHAQ H ABEDLI AL-HADDAD

How to Construct an Asphalt Binder Master Curve and Assessing the Degree of Blending between RAP and Virgin Binders by Abbas Booshehrian; Walaa S. Mogawer, and Ramon Bonaquist

R. West, J. Michael, R. Turochy, and S. Maghsoodloo. Use of Data from Specific Pavement Studies Experiment 5 in the Long-Term Pavement Performance Program to Compare Virgin and Recycled Asphalt Pavements, In Journal of Transportation Record, Transportation Research Board, Washington, D.C., 2011

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

A critical review on limitations of non-linear analysis of multi-storey structures

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Abstract

Structures may suffer severe damages when exposed to extreme loadings such as winds and earthquakes. Earthquake is generated by seismic waves that reach the earth's surface. Damages due to contrasting shaking intensities will be different according to locations. To know the possible damage caused due to earthquake, analysis and design of a building is to be done before the construction of the structures. It has been admitted that, by introducing some sort of nonlinear analysis into the seismic design methodology, destruction of structures can be controlled that is one of the essential design concerns. A non-linear analysis is a form of analysis where a non-uniform relation holds among applied loadings and displacements. It is the most generalized form of analysis and is necessary when loading generates a considerable variation in stiffness. This study aims to present a critical review of the damages of the multi-storey structures in a seismic event, influence of governing input parameters, non-linear analysis (NLA) procedures and limitations of these procedures along with finite element (FE) tools for reliable non-linear analysis. Firstly, damages in structures due to seismic activity reported by researchers around the globe are discussed. Then different FE tools, i.e. ETABS, SAP2000, Perform-3D etc., are compared for non-linear analysis keeping in mind certain constraints (e.g. output, accuracy/precision, time required for analysis, outcome required etc.). The outcome of this review is customization of reliable methods and FE tools against different building types for consultant designers.

Keywords

Non linear analysis, Earthquake, Structures, Irregular

1. Introduction

Pakistan is counted in most seismically active country in world, Earthquakes in this region occur often and cause widespread destruction and loss of human lives. The disastrous earthquake in Muzaffarabad, ziarat, caused majority of the damages in Pakistan. Although there are many other natural disasters, yet earthquakes are highly destructive and unpredictable in all of them. Their damage or collapse can have very adverse and severe effects on the life and on the economy of the distressed areas. Earthquake or lateral forces have devastation effect on structural building all around the world. These forces produce negative aspect; moreover, human safety is the prior aspect. In previous era, buildings were made without earthquake or lateral force consideration. These forces have adamant effect on structural in term of load, causing to increase in the load carrying capacity. However, their provision was not taken into consideration. For this purpose and to save human life, engineers used to calculate these forces in order to restrain the buildings to devastation. To know the damage caused due to earthquake, design and analysis of a structure is to be done before the construction of the structures.

The lateral forces produce shock in the form of seismic waves due to which building collapsed. However, these shocks have adamant effect in developing countries especially in Pakistan. Majority of these forces have influence on North West and area where tectonic plate falls. Majority of building collapsed during 2005 earthquake in which thousands of people lost their lives, houses and even their lands. This was all because of lack of awareness and technical knowledge related to earthquake and its resistant design. It has been documented that damage control is one of the important parameters and it can only be attained by applying nonlinear analysis into the seismic design methodology. During disastrous earthquakes, building design and safety must be given priority concerns and for this purpose behavior of structures needs to be understood, under large inelastic deformations. In this paper, damages of multi-storey building in seismic event, impact of fault mechanism, non-linear analysis procedures, and their limitations have been discussed along with FE tools adopted by researchers around the globe.

2. Damages of multi-storey building in seismic event

In the past two and a half decades, the world has experienced countless earthquakes of greater extents which lead to immense harm to people and wide-ranging physical obliteration. Previous experiences expose that for the identical magnitudes of earthquakes, the damages arose in developed and underdeveloped countries are much more. This might be recognized due to the absence of information and technical knowledge regarding the facets of seismic danger assessments and alleviation. Due to recent severe earthquakes, a lot of studies is required in the development of earthquake resistant structures. Some of the damages are being shown in figure 1. Earthquakes are of considerable importance as they present severe risk to public and welfare in the substantial manner everywhere.

(a) Gul et al 2005

(b) Gul et al 2005

Figure 1: (a) and (b) Damages Observed Due to Earthquake

3. Impact of fault mechanism and seismic zones

Chen and Lui (2006) stated in their book "Earthquake Engineering for Structural Design" that tectonic earthquakes are produced from movement between 15 huge plates containing lithosphere. These 15 plates are driven by movement of the matter in the soil's layer, which in turn is driven by the high temperature produced at the earth's core. The position of original radioactivity of seismic waves is called hypocenter, while the projection on the superficial of the ground directly above the hypocenter is called epicenter. Earthquake ensues at the limits of tectonic plates with utmost frequency. Faults are the visible appearance of the limits amongst adjoining tectonic plates and therefore might be hundreds of miles prolonged. Furthermore, there might be numerous briefer faults similar to or splitting out after a major fault region. Usually, the greater a fault the greater the vibrations it can produce. Faults are characteristically segregated depending upon their motion. Elementary terms contain transform, dip-slip, normal, reverse and thrust faults. Types of faulting of normal, reverse and strike-slip are being shown in figure 2 . Usually, earthquakes will be intense in the locality of faulting, faults that are moving more quickly will be exposed to greater extent of seismicity, and larger faults are more expected than others to produce a larger event.

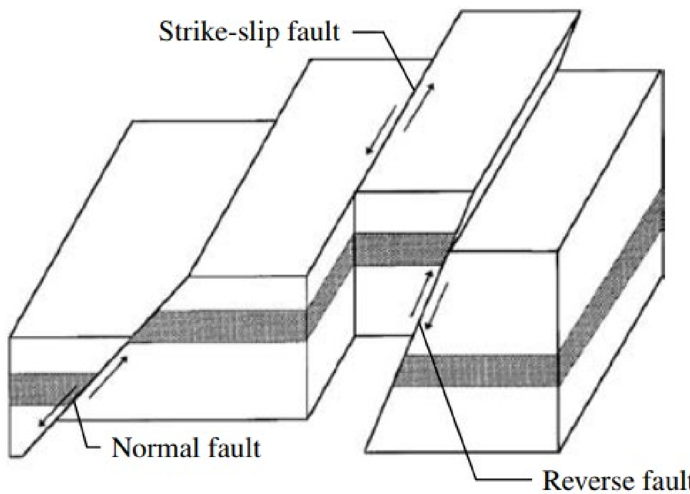

Figure 2: Types of Faulting (Chen and Lui 2006)

4. Non-Linear Analysis Procedures

Chou and Chang (2018) suggested a new method to detect damage based on prediction errors. They utilized the frequency domain approach to identify an input-output model of a safe structure. They converted this model into bank of Kalman estimators and estimated responses depending upon the safe structure. Furthermore, they calculated prediction errors by the assessed responses and deduced into damage indices.

Chou and Chang (2018) presented an innovative method to detect damages depending upon the modal response errors. They distributed a structure into various subsystems in order to assess the subsystem. An input-output frequency approach was utilized to identify subsystem models. Moreover, the standard deviation responses were estimated through a moving window. Sarno (2013) studied the effects of numerous earthquakes on rigid structural response. Rigid constant ductility acceleration, displacement and force reduction spectra were derived for a set of robust motions recorded at five different stations during 2011 earthquake in Japan. He concluded that standardized strength bands for seismic order have presented that the force demand on building can be three times comparative to a single event. However, such demand is considerably affected by the ductility levels, particularly for periods more than 1 seconds. Franke et al. (2019) presented the observations made by UNAM-GEER teams concerning an earthquake of 7.1 magnitude in the Mexico City. It was observed that no structure collapsed due to seismic activity. However, considerable distortions in buildings were observed in numerous structures due to deprived seismic foundation performance specifically in friction piles. Hossienpour and Abdelnaby (2017) examined the non-linear behavior of RC structures under several earthquakes. Two buildings were modelled in Zeus-NL Software. The effect of asymmetry, damage occurred from past events, earthquake path and aftershock polarity etc. They concluded that damage occurred from the past events was more significant in asymmetric structure. Altering the path of the earthquake affects total drift demands and flexible hinges. Moreover, aftershock polarity can considerably alter the drift demands.

Zhao et al. (2015) developed a damage evaluation technique at the small scale only utilizing the data after earthquake polarimetric (HR PolSAR) imagery. The HR imagery offers a flexible possibility for damage evaluation in contrast to SAR data. Vega and Silva (2017) assessed damages due to earthquake considering the attributes of previous events that took place in South America. They utilized Openquake engine software for seismic hazard and risk analysis. They concluded their research stating that unreinforced masonry is the most susceptible in South America. Furthermore, they specified that installing recording stations at definite locations is also beneficial in order to assess the ground shaking. Moreover, a network of seismographs would help in assessing the damage more precisely. Srisangeerthanan et al. (2018) carried out a numerical study for multi-storey segmental buildings on the influence of stiffness of diaphragm and strength of response in case of seismic activity. They concluded that increase in diaphragm results in greater mode contribution and influences the displacement and connection forces of diaphragm. Moreover, multi-storey segmental structures with diaphragms distributed at the limit of being stiff was not affected by greater modes. However, the rigid diaphragm behavior results in greater ground motion which can be controlled through appropriate modification factors. Shrestha and Hao (2018) carried out a study to present the damages occurred from pounding among adjoining structures during Gorkha earthquake. The outcome of their research was that the structures that were exposed to severe pounding had other deficiencies in construction etc. Furthermore, they observed that structures having greater parting gaps from adjoining buildings were also not secured from impacts following collapsed neighboring buildings. Moreover, it was tinted that brick masonry structures were at an immense risk of pounding damage. summary is being attached in Table 1.

Table 1: Summary of NLA methods adopted by researchers

Building description	NLA Methods	Reference
50 storey	Elastic Response Spectrum Analysis, Nonlinear time history analysis	Ozuygur (2016)
5 storey RCC Building	Time history analysis	Gowtham et al. (2017)
High-rise structure with obligue columns	Response spectrum and time history analysis	Hu et al. (2012)
RC structure	Non Linear Analysis	Najafgholipour (2019)
Multi-storeyed RC building	Non Linear dynamic Analysis	Verma et al
Multi-storeyed building	Time History Analysis	Yansiku(2017)
Regular high-rise building	Time history and and static pushover analysis	Wilkinson(2006)

4.1 Limitations of Non-Linear Analysis

If investigation was to be carried out for numerous ground accelerations, there can be a significant disparity in the outcomes. Partially, this is because of the fact that nonlinear behavior is fundamentally subtle to comparatively minor fluctuations in the ground accelerations. Somewhat, it is because of the approaches used to select the ground accelerations. A computer program investigates the nonlinear analysis model, not the real structure. Engineers can carry out modelling with different assumptions and may get diverse outcomes for the identical structure. Similarly, the reaction of the structure might be subtle to the strengths and stiffnesses of its components and the definite properties may not be identified precisely. Capacity design can critically reduce the unreliability. If the strength proportions for some components are greater than 1, re-design can be considered by augmenting component's strength for linear assessments. This could also intensify the stiffness, demanding re-analysis, nonetheless the process generally converges rapidly. For deformation-based design by utilizing nonlinear investigations, the choices can be to intensify the strength, ductility or to alter further properties. It can be more problematic to select a re-design approach, and a greater amount of re-analyses might be essential. These limitations may appear serious but the fact remains that if stiff behavior is existing, nonlinear evaluations are more probable to give reliable design info than a linear evaluation. There might be less unpredictability in the outcome of linear analysis, but this is deceptive since linear analysis is integrally ambiguous. An imperative goal of carrying out a research and code development must be to lessen the unpredictability in nonlinear investigations.

4.2 FE Tools for Non-Linear Analysis

Wilkinson (2006) compared results from both methods of time history analysis and static pushover analysis, the results verified that it is more feasible to perform response history analysis for regular high-rise buildings. Yansiku (2017) suggested time history technique as most reliable method for observing structural responses. Spectral matching process by using ETABs gives better average spectral curves than using seismomatch. Kamath et al (2016) studied performance characteristics of diagrid structures by using nonlinear static pushover analysis as per FEMA 356 guidelines.

Ozuygur (2015) took 50 storey RC building, which have to be constructed in Istanbul and designed for seismic activity by elastic response spectrum analysis and seismic response was checked by time history analysis and software used was PERFORM-3D. Najafgholipour and Arabi (2019) implemented selected model in nonlinear frame analysis software SAP2000. The results of simulation and analysis of three connections with various governing failure modes using the proposed simplified technique showed its capability in capturing different failure modes in the joint region. summary is being attached in Table 2.

Table 2: Summary of FE Tools adopted by researchers

Building description	FE TOOL	Reference
50 storey	Perform-3D	Ozuygur (2016)
5 storey RCC Building	SAP2000	Gowtham et al (2017)
High-rise structure with obligue columns	ETAB,SAP2000	Hu et al.(2012)
RC structure	SAP2000	Najafgholipour (2019)
Multi-storeyed RC building	SAP 2000	Verma et al
Multi-storeyed building	ETABS	Yansiku(2017)
Regular high-rise building	ETABS	Wilkinson(2006)

5. Discussion

A critical review was carried out on the limitations of non-linear analysis of multi-storey structures and previous studies were kept under consideration. Different fault mechanisms were observed i.e. normal fault reverse fault and strike-slip fault. Furthermore, the procedures for non-linear analysis were also reviewed. Among these, FEM modelling of the structure in case of seismic activity serve as the most suitable method for the evaluation of earthquake through non-linear analysis. In addition, various modelling tools were swotted and the most widely used and appropriate tools were observed to be SAP2000 and ETABS.

6. Conclusion

Buildings are not usually designed by doing non-linear analysis. Earthquakes of last some decades warned the human life to take attention towards the issue of designing of earthquake resistant buildings. In this paper, software's and methods of non-linear analysis have been discussed after reviewing different research papers. For high rise buildings only linear analysis is not sufficient as it only deals within elastic region. There is need to study and implement non-linear analysis methods for providing enough strength to our tall structures so that precious lives of people could be saved and this paper will also help in giving recommendations for adopting methods and software's of analysis procedure according to building type on the basis of previous knowledge.

7. References

Chen.W.F and Lui ,E. M.eds. ,(2006).*Earthquake Engineering for structural design*.CRC Press.

Chang, C.M. and Chou, J.Y., (2018). Damage detection of seismically excited buildings based on prediction errors. Journal of Aerospace Engineering, Vol 31(4), p.04018032.

Chou, J.Y. and Chang, C.M., (2018). Decentralized damage detection of seismically-excited buildings using multiple banks of Kalman estimators. *Advanced Engineering Informatics*,Vol *38*, pp.1-13.

Di Sarno, L., (2013). Effects of multiple earthquakes on inelastic structural response. Engineering Structures,Vol 56, pp.673-681.

Franke, K.W., Candia, G., Mayoral, J.M., Wood, C.M., Montgomery, J., Hutchinson, T. and Morales-Velez, A.C., (2019). Observed building damage patterns and foundation performance in Mexico City following the 2017 M7. 1 Puebla-Mexico City earthquake. Soil Dynamics and Earthquake Engineering, Vol 125, p.105708.

Gul F.A, Ali C.M, and Jaafar H.S., (2016). Margalla Towers Islamabad Collapse in Earthquake 2005 in Pakistan. *International Journal of Research in Engineering and Social Sciences,* Vol 6.

Hosseinpour, F. and Abdelnaby, A.E., (2017). Effect of different aspects of multiple earthquakes on the nonlinear behavior of RC structures. *Soil Dynamics and Earthquake Engineering*,Vol *92*, pp.706-725.

Kamath, K., Hirannaiah, S. and Noronha, J.C.K.B., (2016). An analytical study on performance of a diagrid structure using nonlinear static pushover analysis. *Perspectives in Science*,Vol *8*, pp.90-92.

L'Hadj, L.A., Hammoum, H. and Bouzelha, K., (2018). Nonlinear analysis of a building surmounted by a reinforced concrete water tank under hydrostatic load. *Advances in Engineering Software*,Vol *117*, pp.80-88.

Najafgholipour, M.A. and Arabi, A.R., 2019, December. A nonlinear model to apply beam-column joint shear failure in analysis of RC moment resisting frames. In *Structures* (Vol. 22, pp. 13-27). Elsevier.

Özuygur, A.R., (2016), February. Performance-based Seismic Design of an Irregular Tall Building—A Case Study. In *Structures* (Vol. 5, pp. 112-122). Elsevier.

Sekhar, H.V. and Das, T.V., (2017). Analysing The Seismic Behaviour of Set Back Building By Using E-Tabs. *International Journal of Civil Engineering and Technology*,Vol *8*(1), pp.444-451.

Shrestha, B. and Hao, H., (2018). Building pounding damages observed during the 2015 Gorkha earthquake. *Journal of Performance of Constructed Facilities*,Vol *32*(2), p.04018006.

Srisangeerthanan, S., Hashemi, M.J., Rajeev, P., Gad, E. and Fernando, S., (2018). Numerical study on the effects of diaphragm stiffness and strength on the seismic response of multi-story modular buildings. *Engineering Structures*,Vol *163*, pp.25-37.

Villar-Vega, M. and Silva, V., (2017). Assessment of earthquake damage considering the characteristics of past events in South America. *Soil Dynamics and Earthquake Engineering*,Vol *99*, pp.86-96.

Wilkinson, S.M. and Hiley, R.A., (2006). A non-linear response history model for the seismic analysis of high-rise framed buildings. *Computers & structures*,Vol *84*(5-6), pp.318-329.

Yansiku, S.I., (2017). Impact of artificially seismic loading on the response of building structure in various site classifications. *Journal of King Saud University-Engineering Sciences*,Vol *29*(4), pp.302-312.

Zhao, L., Yang, J., Li, P., Zhang, L., Shi, L. and Lang, F., (2013). Damage assessment in urban areas using post-earthquake airborne PolSAR imagery. *International journal of remote sensing*,Vol *34*(24), pp.8952-8966.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Influence of jute fibre and GFRP rebars in compressive and dynamic behavior of prototype thin shear concrete walls

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Abstract
Compressive failure in thin shear wall was observed in several high rise buildings. Experiments done on small thin walls in the past at low uniform compression showed that thin shear walls without confined vertical reinforcement had failed suddenly. The specific aim of this study is to investigate the compressive behavior of prototype thin shear concrete walls having jute fibres and glass fibre reinforced polymer (GFRP) rebars for possible mitigation of abrupt failure. Mix design ratio of PC is 1:2:3:0.6 (Cement: Sand: Aggregates: Water). Jute fibres having length of 5 cm and fibre content of 5%, by mass of cement are used for preparing jute fibre reinforced concrete (JFRC). Two prototype thin shear walls for compression testing is cast, one with plain concrete having steel rebars and one with JFRC having GFRP rebars. Specimens are kept in water at room temperature for 28 days. Dynamic properties are tested according to ASTM C215-02. Compression behavior is studied and compressive strength, energy absorption and conpressive toughness index are determined. Results of JFRC wall are compared with that of plain reinforced concrete wall.

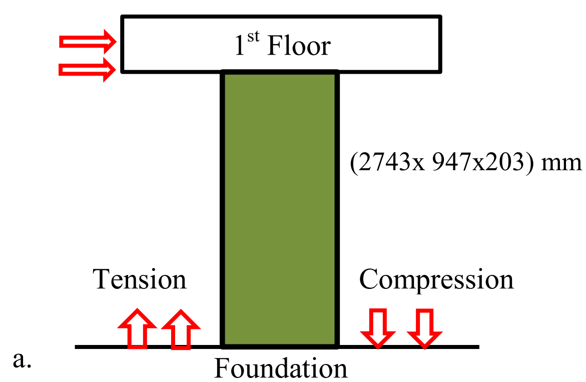
Keywords
Compressive Failure, Glass Fibre Reinforced Polymer Rebars, Jute Fibre Reinforced Concrete, Thin Concrete Walls.

1. Introduction

Concrete shear walls are typically used to resist lateral loading in seismic regions as well as they simultaneously carry vertical loads to the base (Junemann et al. 2016). Adebar (2013) highlighted that many buildings were severely damaged during the February 2010 earthquake in Chile. A common failure in high rise buildings was the compression failure of concrete shear walls. Thickness of the walls ranged from 120 mm to 200 mm. A walls having thickness less than 200 mm can be considered as thin shear wall. Sherstobitoff et al. (2012) highlighted that more than 100 buildings having concrete shear walls were damaged in February 2010 earthquake in Chile. A common type of damage observed was compression failure of thin shear concrete walls at lower stories of buildings. Yathon et al. (2017) highlighted that during 2010 Chile earthquake and 2011 Christchurch earthquake, buildings having shear walls experienced severe damage. The structural configuration of system and non-ductile behavior of thin shear walls led to damage of these buildings. Junemann et al. (2015) highlighted that the damage in medium rise buildings due to thin unconfined walls which were subjected to high axial stresses which increased due to the dynamic effect.

Compression failure was observed when thickness of shear wall is reduced. Adebar (2013) reported that it's a common practice in Chile to make partitions walls between rooms as shear walls. Older buildings had thicker walls while new buildings had thinner walls. Most the damage observed was in thin walls. Rojas et al. (2011) observed that compression in walls of buildings could cause crushing of concrete and buckling of reinforcement. Saatcioglu et al. (2013) highlighted that as 1996 Chilean code did not restrict any structural irregularities. A fifteen story building had a shear wall with an offset at first story towards which it collapsed. The vertical irregularity in this slender shear wall may have caused the compressive stresses to increase at first story level resulting in crushing of unconfined concrete at the ends of wall. Adebar and Lorzadeh (2012) reported that 22 story hotel Grand Chancellor in Christchurch underwent damage beyond repair when its concrete wall in lobby failed in compression. The wall was supporting a transfer girder on sixth story level of building that was cantilever past the concrete wall. To avoid this crushing of concrete, natural fibres in concrete can be used as they create a bridging effect with concrete. Jute fibre has the ability to increase compressive parameters and dynamic properties of concrete. Jute fibre reinforced concrete (JFRC) will create a bridging effect with unstable concrete and glass fibre reinforced polymer (GFRP) rebars will act as corrosion resistant reinforcement.

There is a need to promote cheap and locally available materials in order to save cost without compromising on mechanical properties. Hence JFRC fulfills these standards. Islam and Ahmed (2018) highlighted that the improvement in compressive strength of JFRC increases with the increase of curing days. Omar et al. (2010) investigated the dynamic properties of jute and kenaf fibre. It was observed that under dynamic loading, jute fibre reinforced composite had higher dynamic response than kenaf fibre composites in terms of flow modulus, compression strength and compression modulus. Hussain and Ali (2019) reported that addition of jute fibre in concrete decreased the compressive strength by 6% and increased the compressive toughness of concrete by 4 times of plain concrete. Damping ratio and elastic modulus of JFRC increased by 100% and 68%, respectively, in comparison to plain concrete. Zia and Ali (2017) reported a decrease of 31% in compressive strength of JFRC and an increase of 124% in compressive toughness of JFRC as compared to plain concrete. Developing countries now face the challenge of corrosion of steel rebars. This prompted to look for alternate material to replace steel rebars in future construction. Zang et al. (2019) conducted an experimental investigation on behavior of shear wall using GFRP rebars in seawater sea-sand concrete. It was concluded that GFRP rebars demonstrated adequate deformation and load carrying capacity as compared to same ratio of steel rebars. Replacement of steel bars with GFRP rebars solves the corrosion problem effectively. Mohamed et al. (2014) conducted an experimental investigation by using GFRP rebars in shear wall under lateral cyclic loading. It was observed that there was no permanent deformation in GFRP rebars up to 80% of ultimate capacity. The GFRP walls behaved elastically with realigned cracks. Figure 1 shows the prototype of thin shear concrete wall with simplified boundary condition. Prototype is scaled down by a factor of 1/6 approximately with respect to original dimensions. It is difficult to test full length of shear wall in lab. Hence, a prototype of lower boundary condition is tested in lab.



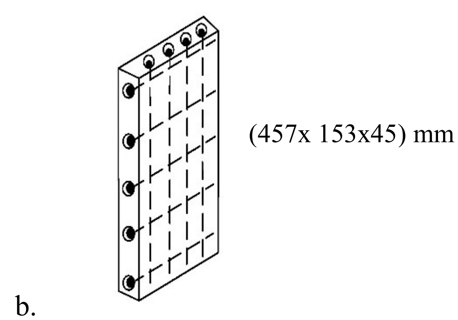


Figure 1:Thin shear concrete wall, a. Full scale shear wall, and b. Scaled down prototype

Current study is done using fibre content of 5% fibre content by cement mass. To the best of author's knowledge, use of jute fibres and GFRP rebars in thin shear wall has not been studied. Hence an experimental study is planned to investigate the potential of JFRC and GFRP rebars. The overall goal of the study is to explore the materials in relation to natural fibres and fibre reinforced polymers rebars (FRP) for use in buildings for enhanced performance. Current study is focused on the influence of jute fibre and GFRP rebars in compressive and dynamic behavior of prototype thin shear concrete walls

2. Experimental Procedures

2.1 Raw Materials

Ordinary Portland cement, water, locally available sand, aggregate, GFRP rebar, steel rebar and jute fibre were used in preparation of the specimen of PC and JFRC. The cement, sand and aggregates were sieved before use as they not properly graded. Difference in PC and JFRC was addition of jute fibre (JF).

2.2 Mix Design and Casting Procedure

For PC, the mix design ratio for cement, sand and aggregates is 1, 2 and 3 respectively, with a water-cement ratio of 0.6. The mix design of JFRC is similar to PC except that 5 cm long jute fibres of 5% by cement mass were added. The electronic drum type concrete mixer is used in preparing PC and JFRC. The slump test value is 4 cm for PC. For preparing JFRC, all materials are added in the mixer in three layers. One third of the total saggregate is placed in the mix followed by one third of JF to that of total quantity of JF for the mix are placed in mixer. Then one third of sand and one third of cement is placed in the mix. This layer-wise addition is done until all he required raw materials for a batch of concrete are added in the mentioned sequence. Then after addition of two third of the water, the drum mixer is rotated for two minutes. Then the remaining water is added in the mixer and the drum mxer is rotated for one to two minutes. A slump test for the JFRC was done before pouring it in moulds. JFRC showed 2 cm slump value. The test is carried out as per specifications of ASTM C143/C143M−15a. Then JFRC is layed in thin wall shuttering, then tamped with the help of tamping rod while a hammer is striked on the outer face of mould for compaction and to remove air voids from the JFRC.

2.3 Mechanical Properties

Cylinders having dimensions of 100 mm x 200 mm are prepared to check the compressive strength of PC and JFRC. Tests are performed according to ASTM standard C138 / C138 – 16 in servo hydraulic testing machine (STM). The compressive strength for PC and JFRC is 14 MPa and 10 MPa, respectively.

2.3 Dynamic Properties

Dynamic testing of prototypes is done according to ASTM standard C215-02. Resonance frequencies (i.e. longitudinal, transverse and torsional frequencies) and damping ratio is determined. Dynamic testing is done prior to the compression testing of prototype thin shear walls.

2.4 Detail and Testing of Prototype Thin Shear Wall

Two prototype thin shear walls are prepared, one for PC and one for JFRC having GFRP rebars in each prototype wall. Same number of rebars are used in both prototypes. Rebars of 6 mm are used. The unit density of PC and JFRC is determined as per ASTM standard C138 / C138M – 16. All specimens prepared are cured for 28 days prior testing. Thin shear wall of PC and JFRC having GFRP rebars are tested in STM for the determination of compressive strength, energy absorbed and toughness index. The compressive strength test is performed as per specification given in ASTM C39 / C39M – 16. The specimens are capped with plaster of Paris for uniform distribution of load prior to testing.

3. Results and Analysis

3.1 Dynamic Properties of Prototype Thin Shear Wall

Table 2 shows the resonance frequencies and damping ratio (ξ) of PRC and JFRC prototype thin shear walls. Resonance shown are longitudinal frequency (f_l), transverse frequency (f_t) and torsional frequency (f_{tr}). There is a decrease in f_l, f_t and f_{tr} of JFRC prototype by 30%, 9% and 38%, respectively, as compared to PRC prototype. An increase of 183% in damping ratio is observed for JFRC prototype as compared PRC prototype.

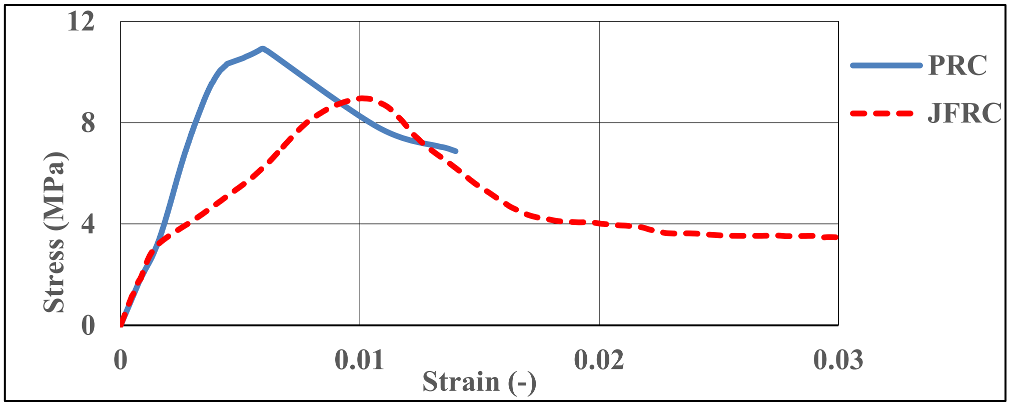
Table 2: Resonance frequencies and Damping ratio

Prototypes	No. of Prototypes	Resonance Frequency (Hz)			Damping Ratio (ξ) %
		f_l	f_t	f_{tr}	
(1)	(2)	(3)	(4)	(5)	(6)
PRC	1	2141	2100	2150	2.3
JFRC	1	1509	1908	1331	6.5

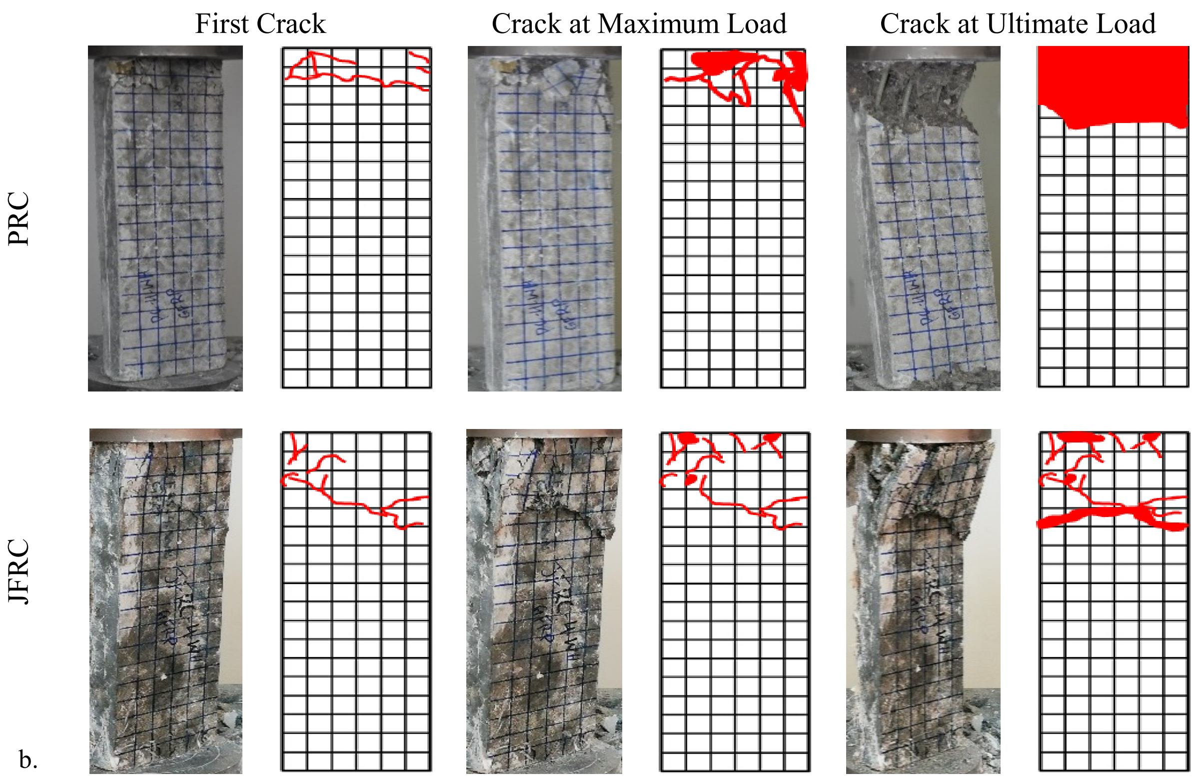
3.2 Compressive Behavior of Prototype Thin Shear Wall

The behavior of PC and JFRC specimens having GFRP rebars during the compression testing (i.e. first crack, cracks at maximum load and cracks at ultimate load) is observed. The stress-strain curve is shown in figure 2(a). Crushing of concrete is observed in the PRC prototype with GFRP rebar as the specimen did not show any resistance after attaining peak load. Hence, the prototype did not show any load carrying capacity after peak loading resulting in low strain as shown in figure 2 (a). The compressive strength of PRC prototype is 10.9 MPa and the strain at this strength is 0.006 whereas JFRC prototype compressive strength is 9.0 MPa with a strain of 0.010. In case of JFRC prototype having GFRP rebars, there is bulging effect at ultimate load i.e. no spalling of concrete. Hence, the prototype is able to withstand loading after attaining peak load resulting in more compressive strain (i.e. ultimate strain) as compared to PRC prototype. High strain at ultimate strength means that JFRC prototype will have a better post cracking behavior as compared to PRC prototypes. This bridging effect is due to the presence of well dispersed jute fibre in concrete prototype.

The Figure 2(b) shows the cracked PRC and JFRC prototypes having GFRP rebars at first crack initiation, cracks at maximum load and cracks at ultimate load. The cracking pattern is almost the same first crack for PRC and JFRC prototype. It is observed that the width, length and depth of the cracks at maximum load in PRC prototype are more as compared to JFRC prototype. Chipping/spalling of concrete was observed in PRC prototype whereas in JFRC prototype, the fibres did not allow any chipping of concrete. There is no spalling of concrete in JFRC prototype even at ultimate load showing a bridging behavior due to presence of equally dispersed fibres, hence avoiding brittle behavior. Spalling of concrete at ultimate loading is observed in case of PRC prototype. After spalling of concrete in PRC prototype, it can be seen that some aggregates are also broken due to low crushing strength. GFRP rebars are visible in PRC prototypes at ultimate cracking stage due to spalling of concrete till the core of the prototype. The prototype of JFRC having GFRP rebars showed strong bonding between rebars and JFRC matrix, as the reinforcement is not exposed at ultimate load. At ultimate load, buckling of GFRP rebars triggers an out of plane failure for both prototypes (i.e. PRC and JFRC). Buckling of reinforcement in JFRC prototype is less evident as compared to PRC prototype shear wall as GFRP rebars are not visible due to chipping of concrete under compression loading.



a.



b.

Figure 2: Behavior of prototype after testing, a. Stress-Strain curve for prototypes of PRC and JFRC with GFRP rebars, respectively, and b. cracked specimens PRC and JFRC with GFRP rebars, respectively

3.3 Compressive Parameters of Prototype Thin Shear Wall

The peak load, compressive strength (σ), peak strain (ε), compressive energy absorption till peak load (CE_m), compressive energy absorption from peak load to ultimate load (CE_u), total compressive energy absorption (TCE) and compressive toughness index (CTI) are given in table 1. A decrease of 17% in compressive strength of PRC prototype is noticed as compared to JFRC prototype thin shear wall. Strain at peak strength, total energy absorbed and toughness of JFRC prototype thin shear wall increased by 67%, 64% and 20%, respectively, as compared to PRC prototype.

Table 1: Peak load, strain, compressive strength, energy absorption and toughness.

Prototypes	Load (kN)	ε (-)	σ (MPa)	CE_m (till P_m) (MJ/m^3)	CE_u (P_m - P_u) (MJ/m^3)	TCE (MJ/m^3)	CTI (-)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
PRC	76	0.006	10.9	0.04	0.07	0.11	2.74
JFRC	62	0.010	9.0	0.06	0.13	0.18	3.29

4. Discussion

The compression failure in thin shear wall occurs due to undamaged concrete becoming unstable. The unstable concrete makes the shear wall fail in a brittle way. The failure happens so suddenly that the sequence of events can't be predicted. The propagation of cracks and pattern of failure of JFRC with GFRP is much different as compared to PRC. In JFRC prototype, no spalling of concrete was observed, concrete remained intact. Reinforcement was exposed in PRC prototype at ultimate load. JFRC prototype had to be broken deliberately into pieces with a lot of effort whereas PRC specimen broke easily after completion of compression testing. This is because of the bridging effect of jute fibre in JFRC prototype. This bridging behavior enhances the ductility of thin shear walls, thus avoiding brittle failure. The prototype are analyzed by comparing (i) damping ratios (ii) compressive strength, (iii) compressive toughness, (iv) strain at peak and ultimate load and (v) cracking pattern. A slight decrease in compressive strength is observed in JFRC prototypes as compared to PRC prototypes. Damping ratio, compressive toughness index and strain is greater for JFRC as compared to PRC prototypes making JFRC prototype ductile under axial loading.

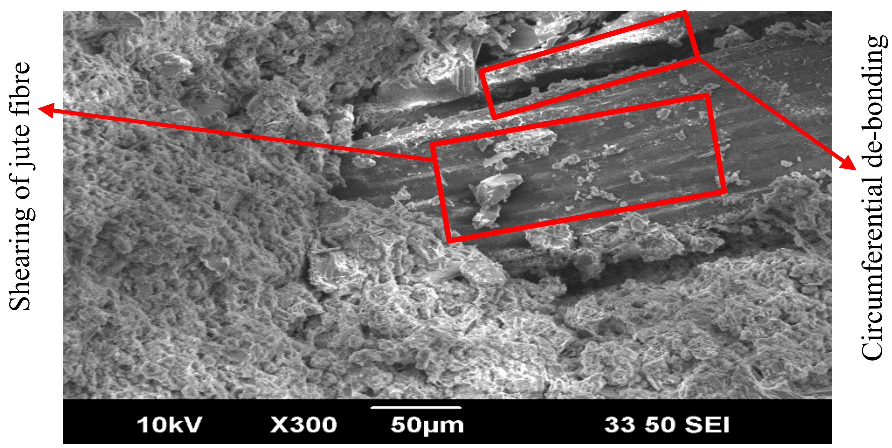


Figure 3: SEM image of JFRC prototype

Figure 3 the SEM image of failure surface of JFRC prototype. It is evident from the image that there is a circumferential de-bonding which shows improper bonding of fibre with rest of concrete matrix. The cavity is not too deep hence, adequate bonding of fibre and concrete matrix is visible in the microstructure. Shearing of jute fibre can be seen in the image. Uniformity of the matrix is visible in SEM image.

5. Conclusion and Recommendations

The study investigates the influence compressive and dynamic properties of JFRC and PRC prototype thin shear walls with GFRP rebars. Following conclusions can be drawn from the conducted study:

- Strain, compressive total energy absorption, compressive toughness index and damping ratio of prototype having JFRC with GFRP rebars increased by 67%, 64%, 20% and 183%, respectively, as compared to PRC prototypes.

- Compressive strength of JFRC prototype decreased by 17% as compared to PRC prototype thin shear wall.

The result shows a positive influence of jute fibre and GFRP rebars in terms of compressive and dynamic parameters. Being a pilot study, results are subjected to simplified boundary conditions. Full scale testing needs to be done to study the in-depth behavior of jute fibre and GFRP rebars in thin shear concrete walls.

6. Acknowledgements

The efforts of Mr. Nasir Shabbir and Mr. Muhammad Junaid are acknowledged for their technical assistance during operating of mixer and servo hydraulic testing machine, respectively. The authors would like to acknowledge the anonymous reviewers for their constructive suggestions.

7. References

Adebar, P. (2013). Compression failure of thin concrete walls during 2010 Chile earthquake: lessons for Canadian design practice. *Canadian Journal of Civil Engineering*, *40*(8), 711-721.

Adebar, P., & Lorzadeh, A. (2012, September). Compression failure of thin concrete walls. In *th world conference on earthquake engineering*.

Hussain, T., & Ali, M. (2019). Improving the impact resistance and dynamic properties of jute fiber reinforced concrete for rebars design by considering tension zone of FRC. *Construction and Building Materials*, *213*, 592-607.

Islam, M. S., & Ahmed, S. J. (2018). Influence of jute fiber on concrete properties. *Construction and Building Materials*, *189*, 768-776.

Jünemann, R., de la Llera, J. C., Hube, M. A., Vásquez, J. A., & Chacón, M. F. (2016). Study of the damage of reinforced concrete shear walls during the 2010 Chile earthquake. *Earthquake Engineering & Structural Dynamics*, *45*(10), 1621-1641.

Jünemann, R., de La Llera, J. C., Hube, M. A., Cifuentes, L. A., & Kausel, E. (2015). A statistical analysis of reinforced concrete wall buildings damaged during the 2010, Chile earthquake. *Engineering Structures*, *82*, 168-185.

Mohamed, N., Farghaly, A. S., Benmokrane, B., & Neale, K. W. (2014). Experimental investigation of concrete shear walls reinforced with glass fiber–reinforced bars under lateral cyclic loading. *Journal of Composites for Construction*, *18*(3), A4014001.

Omar, M. F., Akil, H. M., Ahmad, Z. A., Mazuki, A. A. M., & Yokoyama, T. (2010). Dynamic properties of pultruded natural fibre reinforced composites using Split Hopkinson Pressure Bar technique. *Materials & Design*, *31*(9), 4209-4218.

Saatcioglu, M., Palermo, D., Ghobarah, A., Mitchell, D., Simpson, R., Adebar, P., Tremblay, R., Ventura, C., & Hong, H. (2013). Performance of reinforced concrete buildings during the 27 February 2010 Maule (Chile) earthquake. *Canadian Journal of Civil Engineering*, *40*(8), 693-710.

Sherstobitoff, J., Cajiao, P., & Adebar, P. (2012). Repair of an 18-story shear wall building damaged in the 2010 Chile earthquake. *Earthquake spectra*, *28*, 335-348.

Yathon, J., Adebar, P., & Elwood, K. J. (2017). A detailed inventory of non-ductile concrete shear wall buildings. *Earthquake Spectra*, *33*(2), 605-622.

Zhang, Q., Xiao, J., Liao, Q., & Duan, Z. (2019). Structural behavior of seawater sea-sand concrete shear wall reinforced with GFRP bars. *Engineering Structures*, *189*, 458-470.

Zia, A., & Ali, M. (2017). Behavior of fiber reinforced concrete for controlling the rate of cracking in canal-lining. *Construction and Building Materials*, *155*, 726-739.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

IDENTIFICATION OF CRACKS IN CONCRETE BY VISUAL INSPECTION AND RECOMMENDATION FOR THEIR REPAIR - CASE STUDIES

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ABSTRACT

Cracks in concrete structures occur due to different reasons. During the service life of a concrete structure, its durability is influenced by a number of factors viz. temperature variations, seasonal climatic changes, freeze-flaw cycles, exposure to water, flooding, earthquake etc. Additionally, changes in service loads, deficiencies in either design or field execution practices, result in damage to concrete. The extent and type of concrete distress is directly proportional to the severity of both natural and human-generated deteriorating exposure conditions. Success of concrete revolves essentially around three major features: diagnostic analysis of the cause of damage, selection of suitable products for repair and the availability of an easily adaptable technique for the application of repair methodology.

This paper discusses basics about cracks and their repair materials. The paper includes some real case studies where cracks were identified by visual inspection, their causes have been discussed and suggested preventive measures, techniques and methods to repair the cracks along with estimation of cost for their repair.

Keywords
Concrete, Cracks, Visual Identification, Repair, Rehabilitation.

1. INTRODUCTION

A complete or incomplete separation of either concrete or masonry into two or more parts produced by breaking or fracturing, is called crack. Cracks in concrete can be categorized as structural or non-structural. Structural cracks are mainly a result of defective design, flawed construction or overloading whereas non-structural cracks are caused due to internally induced stresses [1]. There are multiple scenarios that might lead to crack formation in concrete however cracks need to be repaired for building safety, better aesthetics and for prevention from failure. There are a number of chemicals and materials for repair of cracks. These materials can be simple applications or involve complex repair procedures for non-structural and structural cracks, respectively.

The paper discusses basics about cracks and their repair materials. The paper includes some real case studies (around Karachi) where cracks were identified by visual inspection, their causes have been discussed and suggested preventive measures, techniques and methods to repair the cracks along with estimation of cost for their repair.

Field work was conducted at the following five locations.

- Natha Khan Bridge, Karachi.
- Millennium Mall Bridge, Karachi.
- Block-B Ashyana building north karachi.
- Karachi Center New-Town Jail chorangi, Karachi.
- Sindh Muslim Arts and Commerce College, Karachi.

2. TYPES OF CRACKS IDENTIFIED DURING FIELD WORK

During the field work six different types of cracks were identified at different locations/structures in Karachi. High quality images were recorded, and then the author analyzed the cracks for their causes. After analyzing the author categorized cracks as structural and non-structural cracks.

Structural cracks are those which appear on the structure due to the some applied load, over loading and poor soil bearing. These cracks are dangerous because when structural cracks appear on any structure then the life span of building decreases rapidly. On the other hand, Non-structural cracks are not dangerous for the building, but if a non-structural crack left unattended can become the cause of serious structural damage.

Following are the Structural and Nonstructural cracks, which the authors identified during field work.

Structural Cracks:	Non Structural Cracks:	
1. Deformation	1. Delamination	3. Shrinkage
2. Joint-spall	2. Spalling	4. Early Thermal Contraction

3. CASE STUDIES OF DIFFERENT NONSTRUCTURAL CRACKS

3.1 Delamination Crack at Slab of 1ST Floor Classroom of Sindh Muslim Arts and Commerce College, Karachi. (Fig: 1)

Figure 1: Delamination crack

3.1.1 Causes of delamination cracks:
Due to moisture attack and premature stripping of finishing works are likely causes of delamination crack [2].

3.1.2 Suggested remedial measures [3]:
Step-1: Drill multiple 3/8 inch (1 cm) holes into the concrete near the crack. (fig: 3.1.2.1)

Step-2: Remove dust with a vacuum and make sure the surface does not have any type of moisture, laitance or any carbonated paste.

Step-3: Inject low viscosity epoxy adhesive with an epoxy injection applicator till the epoxy comes out of the hole. While implanting the material avoid high pressure injection. (fig: 3.1.2.2)

Step-4: After filling, the holes blend into the surrounding concrete and then clean the surface. (fig:3.1.2.3)

3.1.3 Suggested materials for repair:
The material Sikadur-52 ® (low viscosity epoxy adhesive) is available in local market and is packed in cans, buckets and packets. The coverage rate of Sikadur-52 ® is approximately 3 to 5 m^2/liter/coat on surface [4]. (fig:3.1.3.1)

Figure 3.1.2.1

Figure 3.1.2.2

Figure 3.1.2.3

Figure 3.1.3.1

3.1.4 Cost of repairing a typical delaminated area: (In Pak rupees. conversion: 1 Pak Rupee = 0.0064 US $). (Table: 3.1.4.1)

Table 3.1.4.1

Description of Material	Sikadur-52 ®
Cost of material (Rs.)	3600/liter for 3 square meter of area.
Approximate labor cost for skilled labour	@ Rs.1600 per day of 8 hours (one labour hired for 4 hours)
Approximate cost of misc. supplies, consumables tools,	200/ m^2
Total estimated cost for repair (Rs)	2200/m^2

3.2 Spalling Crack at 1st Floor Corridor of Sindh Muslim Arts and Commerce College, Karachi.

(Fig: 2)

Figure 2: Spalling Crack

3.2.1 Causes of Spalling cracks:
The steel is attack by moisture, which migrate through concrete and corrode the steel reinforcement so the Corrosion of reinforcement results in spalling [2].

3.2.2 Suggested Remedial Measures [5]:
Step-1: At the spalled areas, concrete must be removed so corroded steel bars exposed. (fig: 3.2.2.1)
Step-2: Scrape and clean the exposed steel bars and use a wire brush to remove any rust. (fig: 3.2.2.2)
Step-3: Steel bars must be coat at least 2 times of anti-rush paints.
Step-4: A bonding agent must be applying to the affected surface, before patching the area to ensure proper adhesion, using polymer modified cement mortar patch up the area. (fig: 3.2.2.3)
Step-5: Paint over the patched area to match the rest of your ceiling. (fig: 3.2.2.4)

3.2.3 Suggested Materials for Repair:
The material Sika-MonoTop-610is available in local market and is packed in packets. The coverage area of Sika-MonoTop-610® is approximately 4 kg/m2 @ 2mm thick on surface. (fig: 3.2.3.1)

Figure 3.2.2.1

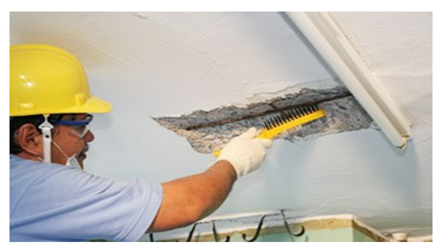

Figure 3.2.2.2

Figure 3.2.2.3

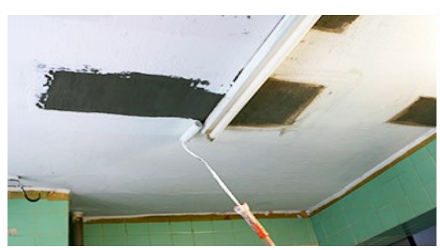

Figure 3.2.2.4

Figure 3.2.3.1

3.2.4 Cost of Repairing a Typical spalled Area. (Table: 3.2.4.1)

Table: 3.2.4.1

Description of Material	Sika-MonoTop-610 ®
Cost of material (Rs.)	300/kg for 0.25 square meter of area at 2mm
Approximate labor cost for skilled labour	@ Rs.1600 per day of 8 hours (one labour hired for 4 hours)
Approximate cost of misc. supplies, consumables tools,	200/m^2
Total estimated cost for repair (Rs)	2200/m^2

3.3 Shrinkage Crack at Column Cop of Millennium Bridge Karachi. (Fig: 3)

Figure 3: Shrinkage Crack

3.3.1 Causes of shrinkage cracks:
Due too much load and vibration, or some time due to impropriate curing [2].

3.3.2 Suggested remedial measures:
Step-1: The first step is to clean the cracks that have been contaminated. (fig: 3.3.2.1)
Step-2: **Now seal crack with mortar,** keep the epoxy from leaking. (fig: 3.3.2.2)
Step-3: **Now drill holes into the crack and fit the pipes. (fig: 3.3.2.3)**
Step-4: **Now inject the epoxy with pressure pump or hydraulic pump. (fig: 3.3.2.4)**
Step-5: **After the injected epoxy has cured, remove the surface seal with grinder. (fig: 3.3.2.5)**

3.3.3 Suggested materials for repair:
The material Epoxy injection (Chemdur-52) is available in local market and is packed in packets and tubes. The coverage area of Chemdur-52 is approximately 3 to 4 m^2/liter/coat on surface. (fig: 3.3.3.1)

Figure: 3.3.2.1

Figure: 3.3.2.2

Figure: 3.3.2.3

Figure 3.3.2.4 **Figure 3.3.2.5** **Figure 3.3.3.1**

3.3.4 Cost of repairing a typical shrinkage area: (Table: 3.3.4.1)

(Table: 3.3.4.1)

Description of Material	Epoxy injection (Chemdur-52)
Cost of material (Rs.)	2200/lit for 1m^2/mm of cracks area.
Approximate labor cost for skilled labour	@ Rs.1600 per day of 8 hours (one labour hired for 2 hours)
Approximate cost of misc. supplies, consumables tools,	500/m^2
Total estimated cost for repair (Rs)	3100/m^2

3.4 Early Thermal Contraction Crack at Wall of Ground Floor of Block-B Karachi Center New-Town Jail chorangi, Karachi. (Fig: 4)

Figure 4: Early Thermal Contraction Crack

3.4.1 Causes of early thermal contraction cracks:
Early thermal contraction cracks occur due to some reasons like temperature rising, coefficient of thermal expansion of the concrete and restraint to movement (internal / external) [6].

3.4.2 Suggested remedial measures:
Step-1: Widening the crack should be the first step creating a keyed surface where the concrete patch can be placed. (fig: 3.4.2.1)
Step-2: Using a wire brush to remove the all debris.
Step-3: Mixing the Portland cement with water that will be used to repair crack. (fig: 3.4.2.2)
Step-4: Before patch the mortar, wet the crack with water.
Step-5: Now patches the cement mortar on the whole area of crack. (fig: 3.4.2.3)
Step-6: At the last finish and smooth the patch cement mortar with the trowel.

3.4.3 Suggested materials for repair:

The material Portland cement is available in local market and is packed in packets. The coverage area of Portland cement is approximately 50kg/0.0347m^3 on surface. (fig: 3.4.3.1)

Figure 3.4.2.1

Figure 3.4.2.2

Figure 3.4.2.3

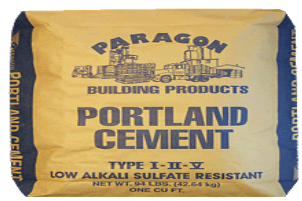

Figure 3.4.3.1

3.4.4 Cost of repairing a typical early thermal contraction area: (Table: 3.4.4.1)

(Table: 3.4.4.1)

Description of Material	Portland Cement
Cost of material (Rs.)	620/bag of 50kg and for 1m^{2}5 ***kg***/m^2/mm .
Approximate labor cost for skilled labour	@ Rs.1600 per day of 8 hours (one labour hired for 4 hours)
Approximate cost of misc. supplies, consumables tools,	200/m^2
Total estimated cost for repair (Rs)	1100/m^2

4. CASE STUDIES OF DIFFERENT STRUCTURAL CRACKS

4.1 Deformation Crack at the Middle Column at 2nd Floor Flat A-12 Block-B Ashyana Building North Karachi. (fig: 1)

FIGURE 1: Deformation Crack

4.1.1 Causes of deformation cracks:
Due to overloading of live and dead loads, thermal expansion of the concrete, shrinkage of concrete and temperature variations [7].

4.1.2 Suggested remedial measures:
Step-1: Drilling holes at the surface of deformation**.** (fig: 4.1.2.1)
Step-2: Clean the drilled holes by blowing air through a tube inserted in the holes. (fig: 4.1.2.2)
Step-3: Inject epoxy with the help of nozzles inserted inside the drilled holes and to fill it from bottom up to the half of the holes. (fig: 4.1.2.3)
Step-4: Insert the reinforcement bars around the columns and allow the epoxy adhesives to cure. (fig: 4.1.2.4)
Step-5: Install and fix the formwork around the reinforcement bars. (fig: 4.1.2.5)
Step-6: At last, the cavity is to be filled through pumping. (fig: 4.1.2.6)

Figure 4.1.2.1

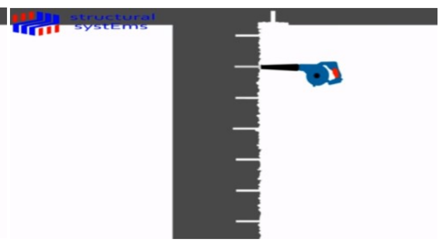

Figure 4.1.2.2

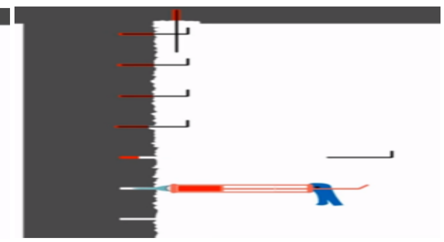

Figure 4.1.2.3

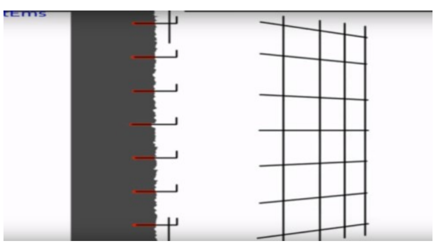

Figure 4.1.2.4

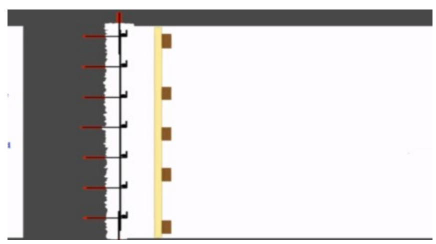

Figure 4.1.2.5

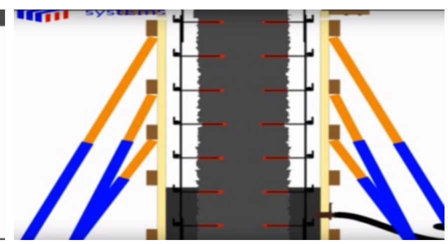

Figure4.1.2.6

4.1.3 Cost of Repairing a Typical Deformed Area: (In Pak Rupees. Conversion: 1 Pak Rupee = 0.0064 US $). Table: 4.1.3.1

(Table: 4.1.3.1)

Description of Material	Bonding epoxy, steel mash, steel bars and
Cost of material (Rs.)	1650/kg epoxy for m^2/mm area. Steel cost is approx. 2000/m^2
Approximate labor cost for skilled labour	@ Rs.1600 per day of 8 hours (two labour hired for 4 hours)
Approximate cost of misc. supplies, consumables tools,	1000/m^2
Total estimated cost for repair (Rs)	6250/m^2

4.2 Joint Spall Crack at Railing of Natha Khan Bridge Karachi. (fig: 2)

FIGURE 2: Joint Spall Crack

4.2.1 Causes of joint spall cracks:
Occurs as a result of overloading of slab edges of joints on vehicular traffic surfaces [2].

4.2.2 Suggested remedial measures [8]:
Step-1: At 90 degrees drill holes which can intersects the crack plane. (fig: 4.2.2.1)
Step-2: Place a reinforcement bar to fill the hole. (fig: 4.2.2.2)
Step-3: Inject epoxy in the hole to bond the bar. (fig: 4.2.2.3)
Step-4: Attach the wall plate and tightened it. (fig: 4.2.2.4)
Step-5: Epoxy is use to bond the bar with the walls of the hole, to fill the crack. It also brings the crack surface back together in monolithic form and strengthens the section

Figure 4.2.2.1

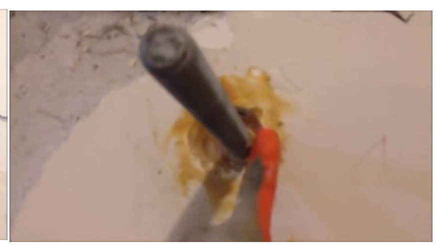

Figure 4.2.2.2 & 4.2.2.3

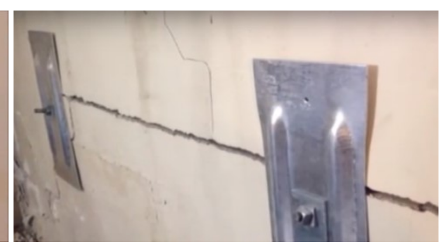

Figure 4.2.2.4

4.2.3 Cost of Repairing a Typical Joint-Spalled Area: (In Pak Rupees. Conversion: 1 Pak Rupee = 0.0064 US $) Table: 4.2.3.1

TABLE 4.2.3.1

Description of Material	Epoxy and steel bar + plate
Cost of material (Rs.)	1650/lit for square meter of area. And steel bar and plate approx. 1000.
Approximate labor cost for skilled labour	@ Rs.1600 per day of 8 hours (one labour hired for 4 hours)
Approximate cost of misc. supplies, consumables tools,	$500/m^2$
Total estimated cost for repair (Rs)	$3950/m^2$

5. CONCLUSIONS

The health of concrete structures largely depends on its usage and vigilant maintenance program. A non-structural crack if left unattended can become the cause of serious structural damage. It is very important for engineers to learn the science of crack propagation and be able to carry out repair works timely and economically. The potential causes of crack can be controlled if proper consideration is given to construction materials and techniques used during construction.

A number of products and chemicals are available in the market that is recommended for repair of different cracks in concrete. These products are by and large imported chemicals. It would become very risky and uneconomical to carryout repair works without understanding the correct procedures for applying the repair materials. The project team has documented different repair procedures with approximate estimated costs that are likely to incur for repairs by using different techniques and materials.

The author hopes the field work covered in this paper will be handy and useful information for maintenance engineers for concrete structures. The author recommends that a comprehensive Research and Development program must be initiated by tripartite collaboration of the academia, industry and entrepreneurs for developing local products for repair of concrete.

6. ACKNOWLEDGEMENT

I wish to express my sincere appreciation to my Professors, Engr. Rukhsana Rahooja and Assoc.Prof. Mah Talat Mirza who supported and gave advice at every stage of paper, I also thanks to my friends Hamza Qureshi and Saeed Gabol for their collaboration in this project.

7. REFERENCES

1. ACI 224.1R-07-Causes, Evaluation, and Repair of Cracks in Concrete Structures.
2. 224R_01-CONTROL OF CRACKING IN CONCRETE STRUCTURES.
3. Staff, Concrete Construction (1998, July 1). Repairing Dry-Shake Hardeners Delamination. Retrieved *from: https://www.concreteconstruction.net/how-to/repairing-dry-shake-hardener-delaminations_o.*
4. Issa, C. A., & Debs, P. (2007). Experimental study of epoxy repairing of cracks in concrete. *Construction and Building Materials*, *21(1), 157-163.*
5. Spalling Concrete. (n.d.). Retreived from: https://www.hdb.gov.sg/cs/infoweb/spalling-concrete.
6. Early-Age Cracking in Concrete: Causes, Consequences, Remedial Measures, and Recommendations, Md. Safiuddin 1,*, A. B. M. Amrul Kaish 2, Chin-Ong Woon 3 and Sudharshan N. Raman 3,4,*, *Published: 25 September 2018, Journal of Applied Sciences*
7. Mirza, H.W. (n.d.). Recent Developments in Products & Technologies for Repair & Retrofitting of *Concrete Structures.*
8. Lin, W. T., & Cheng, A. (2012). Abrasion resistance of cement-based composites. In *Abrasion Resistance of Materials*. *InTech.*

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

FINITE ELEMENT MODELLING OF UPLIFT IN STRUCTURES - A REVIEW

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Abstract

Lateral forces in the structure can be reduced by providing uplift mechanism in vertical elements. This is a new trend which also needs to be studied in detail, particularly through finite element modelling. The aim of this paper is to review the available options of modelling uplifts in structures. The development of uplift concept is briefed along with its expected outcome. The available options of modelling uplifts in FE software are elaborated along with their limitations. The behaviors of structures with uplift obtained from FEM by other researchers are explained in detail. Also, recommendations made about modelling uplift in order to have precise behavior are highlighted. The outcome of this study is to select the most feasible FE technique for modelling uplifts in structures to obtain reliable and robust results.
Keywords: uplift mechanism, limitations, multiple uplifts, finite element technique

1. Introduction

The different techniques about seismic design structures are develop across the world nearly 100 years ago to protect the structures and safe lives under large earthquakes. The techniques are developed to provide the plastic deformation in structures and dissipate maximum energy and also develop with respect to the economic and social requirements and sustainable development goals. These techniques can not restrict the earthquake disaster but can minimize their damages. The naturally occurring events cannot be stopped by humans but can lived with it by different solutions. The structures are designed in such a way that their plastic deformation occur in large earthquakes and remain in elastic for small or moderate earthquakes to avoid their destruction and protect the human lives (J. Takagi et. al 2017).

In these techniques, a new one technique under observation now a days is uplift which is given to the structure to have maximum energy dissipation under strong ground motions. The activated forces in structures during earthquake can be reduced through allowable uplift in structures as a mitigation measure (Qin and Chouw 2010). In past studies, it has been found that the multiple uplifts can be used in structures to minimize seismic damages. Column base hinging was not seen in modern steel structures whereas it could have reduced the earthquake effects significantly (Macrae et al. 2015). Multiple uplifts can be used in the structure to develop flexibility. In the previous studies, it was found that the flexibility of the column base shows a significant decrease in earthquake forces. A reduction of about 21.3% of displacement was recorded in prototype structure through uplift mechanism (Qin and Chouw 2010).

Finite element is a numerical approach for analyzing problems to solve them. The influence of uplift on the seismic behavior of structure can be explained through numerical analysis. The Finite Element modelling is done to analyze the 3D seismic response of the structure with multiple uplifts using different software's ETABS, SAP2000, STAAD Pro, ABAQUS, Perform 3D, ANSYS Civil, RISA, Tekla, SCIA, LUSAS and RFEM etc. The approach to finite element analysis enables structures to be analyzed and the appropriate parameters to ensure their safety to be determined. The damage level of structures due to earthquake was determined by using finite element modelling in Indonesia (R. suryanita et. al 2019). The population increases around the world as the time passes so we need efficient, cost-effective, sustainable and quick design of modern infrastructures. ETABS and SAP are the most popular finite-element modelling programs for complex and multi-story structures. Statistical experiments perform to evaluate the multi-story shear wall seismic performance of an adjacent structure in which uplift was considered in the base using SAP2000 program (Mori et al. 2008).

2. Uplift- a new mechanism to reduce lateral forces

Sarrafzadeh et al. (2014) experimentally studied the effect of uplift, plastic hinge and soil nonlinearity. A single degree of freedom scaled structure was used for the study. Acceleration, displacement and energy was recorded. The seismic energy was observed to have reduced in the structure due to plastic hinge development with fixed base. Uplift is caused due to plastic deformation of soil, which together dissipates more energy than structures with fixed base and plastic hinges.

Cui et al. (2016) experimentally studied by using shaking table the seismic behavior of self-centering of reinforced concrete. From the past history of earthquake events special attentions was paid on the limitations of seismic lateral load resisting systems, and experienced heavy damages and large residual deformations which results economic loss and difficult to repair as well as costly. In prototype structure, column and base joints were free for uplift mechanism, and the joints on the column-beam were remained opened and performed test by using shake table. Accelerometers position was decided in each corner for the recording of lateral deformation. To measure the gap openings between base, column and beam joints, linear voltage displacement transducers were at the ends of beams and columns. The opening is uplifting of beam opening and as well as column opening. Gap openings are directly dependent on intensity of earthquake and can be detected after the basic intensity earthquake.

Ceccotti et al. (2013) conducted a test on storey that consisted of seven-storey and it was full-scale structure. 3D shaking table was used for the purpose of testing this prototype. The purpose was to study the behavior of structure under harmonic loading. Mostly affected country by earthquake that is Japan to minimize the effects of earthquake on structures. On a three-storey building in Tsukuba, Japan; A 1D full-scale test was performed by using shake table. The prototype was properly designed using simple lateral force method. The prototype contained 265 equipment for measuring displacements and accelerations. Inter-storey drift and uplift are measured. It was noticed that in the upper stories maximum acceleration and displacement was found.

Antonellis et al. (2015) conducted a test by using the shake table which was large in size and its performance was also high relatively, was placed outdoor for Earthquake Engineering. Total three number of tests were performed on that shake table. Six historical ground motions were included in that test and concluded peak drift. Bridges are specially designed where safety should be 100 percentage, it is quite expensive and time-consuming when it can be repaired after earthquake. The prototype geometry was based on fixed base which is of full scale of 1.2 meters diameter supporting a column and having seismic weight. When test was performed there was produced ground motions than there will be uplift produced in the base and column of prototype and whole structure will be affected and a lot of damages will occur. To identify the total amount of energy decapitated during test and to take economic measurements.

Qin et al. 2013 investigate the steel prototype of two storey with fixed base and allowable uplift on rigid base. This prototype is 3m in height and 25m^2 floor area for every storey. A shake table is used to perform the test and the time history graph and response spectra is drawn. In his investigation it is clear that the structural damage can be reduced by uplift. The structure with uplift shows the less plastic behavior with respect to fixed base. Due to uplift, the displacement during load case 1 reduced from 11.4mm to 5.2mm, just like this for load case 2 the permanent displacement of 5mm was noticed. It is clear from this test that 55% decreases in residual horizontal top displacement and for load cases 1 and 2 is 75%, respectively. Due to minimizing plastic deformation in structures it comprises more earthquake resistant capacity and thus safety of building. This investigation concludes as a (1) It is completely assured that plastic hinge development can reduced by uplift (2) Less vibrations are produced due to uplift as compared to the fixed base. (3) During earthquake, Structural response can not only reduce by the plastic deformation of soil but also footing vertical displacement.

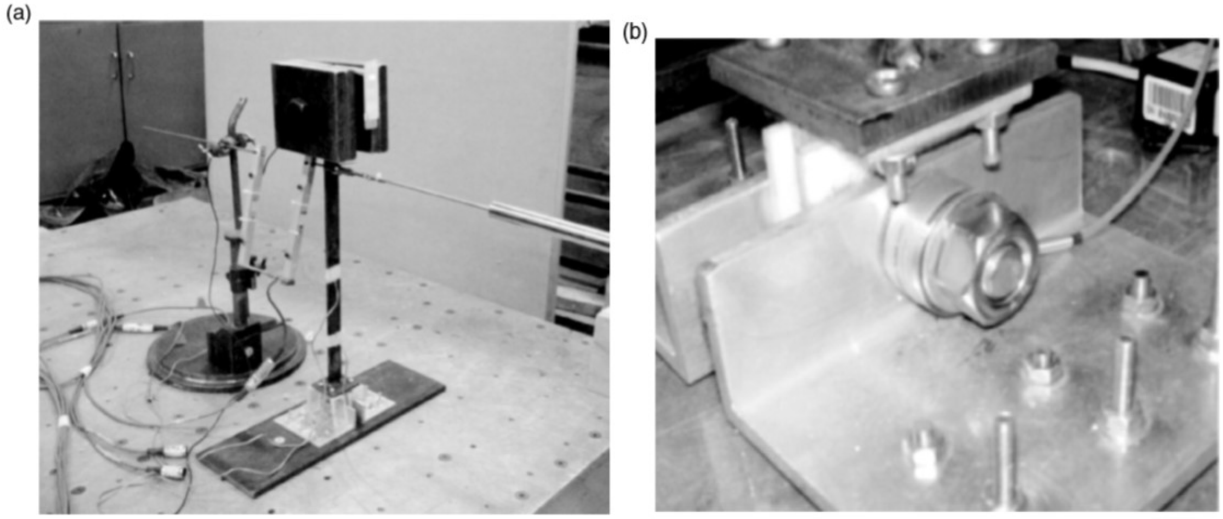

Figure 1. Scaled down SDOF model: (a) Model on fixed base; and (b) column base with artificial plastic hinge (Qin et al. 2013)

Wiebe et al. (2012) Control rocking steel frames built to make it efficient in a way to minimize the structural damages and residential destructions that are due to the seismic force resisting systems. The eight-storey prototype structure is designed and low amplitude shake table for analysis. Results shown that the excessive displacements are cause due to the better capacity design by reducing variability of peak seismic force demand. Low amplitude seismic tests on shake table, the deformations in the structure small.

3. Different options to model uplift in FE

Khan et al. (2014) performed nonlinear analysis against seismic load to study the effect of damper in a multistory RC structure. The dampers are being used in buildings to reduce the effect of earthquake forces. The particular study was done through the finite element nonlinear time-history method using a FEAP SAP2000. Comparative study was done between the applications of dampers up to fifth floor only and up to ninth floor. It was concluded that the dampers can be used to enhance the performance of structure as they can dissipate more energy during earthquakes.
Mori et al. (2008) performed experimental and statistical experiments to evaluate the multi-story shear wall seismic performance of an adjacent structure in which uplift was considered in the base. SAP2000 program has been used for the numerical modeling and analysis of the restore force characteristics. The Uplift mechanism has been used by the system gap element and the results were obtained. The structure is check with uplift at foundation and fixed foundation by loading. On first loading in which uplift is given at the foundation shows only the few cracks on the structural wall. Now load is applied on fixed foundation it shows complete failure of structural wall.
Grange et al. 2009 uses the macro-element for dynamic soil-structure interaction (SSI). A new advanced numerical tool is used to provide seismic loading on structure with shallow foundations. It usually shows the uplift of the foundation, plasticity of the soil, radiation damping and P–θ effects. The macro-element for dynamic loadings is introduced hereafter considering radiation damping, but also P–θ effects (second order effects due to the rotation of the foundation). It is implemented into FEDEASLab, a finite element MATLAB toolbox. The classical theory of plasticity is used to reproduce the non-linear behavior of soil and the uplift of the foundation. The theory of multi-mechanism is used to combine the plasticity and uplift. The ultimate bearing capacity of the foundation fails under combined loads are described by interaction diagram. The experimental results of the structure campus IV shows the performance of the macro-element and the capability of macro-element is shown by the parametric study on the behavior of seven story building. This study also presents that how visco-plasticity theory relate with the macro-element which deals with the uplift of the foundation and plasticity in the soil. At the bottom of numerical

models, a macro-element is introduced to simulate the effect of SSI. It is clearly identified that the global forces and the damage can be reduced by isolate the structure using SSI. Due to the change of position of plastic hinge and by increasing the effect of higher modes, the intro of different local behavior can be obtained and also it is found that the lateral displacement cannot be increase by decreasing the stiffness of soil.

Vassiliou et al. 2016 investigate the seismic response of rocking structures and deformable rocking bodies using a new finite element model. In the model beam elements which show the rocking body and at the ends of the rocking body, rocking surfaces are shown by cross section elements of zero-length fiber. The energy dissipation during rocking motion is model by using the dissipative time step integration numerically by Hilber–Hughes–Taylor. The vertical and horizontal displacements of a rigid rocking block model is proofed through prediction and proved against the analytical Housner model solution for the rocking response of rigid bodies which are under strong ground motion excitation. The modeling of the rocking response of deformable bodies and structures are facilitate by a dissipative model of the ground under the rocking surface. A deformable rocking frame structure to symmetric and anti-symmetric under Ricker pulse ground motion excitation have overturning and uplift rocking response spectra which is checked by this model. It is investigate that the stability does not jeopardize the deformability of the columns of the rocking frame under Ricker pulse ground excitations. In some cases the rigid counterpart is not stable as compare to deformable rocking frame. Due to small remaining displacements and small forces transfer to foundations, the structures are design with uplift at the base and sustain its motion of rocking. Flexible and rocking frames are two similar frames with equal safety factor against uplift, the flexible frames have more stability against overturning.

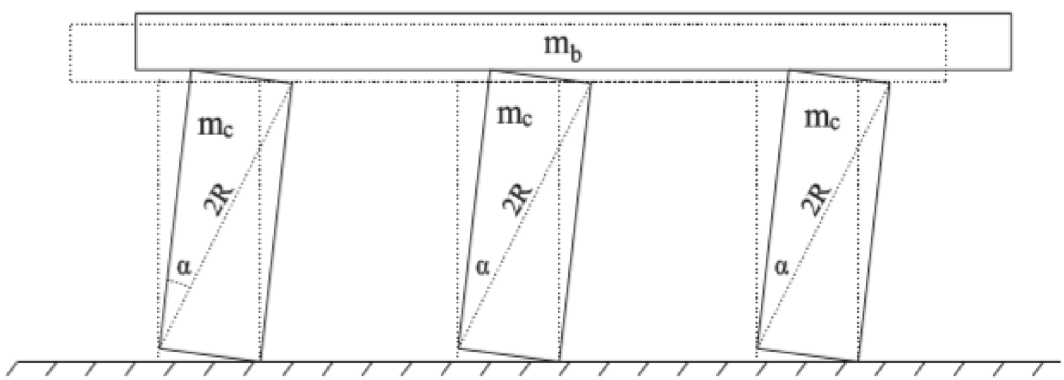

Figure 2. Rocking frame of rigid three-column (Vassiliou et al. 2016)

Rinaldin et al. 2013 a numerical model of Cross-laminated (X-lam) timber building is made to explain it cyclic response and estimate it dissipative capacity. The model is made in such a way that metal hold-downs, angle brackets and screwed connectors used for connections between the foundation and panel with non-linear hysteretic multi-spring elements are used for the strength interaction between different degrees of freedom. The elastic shell elements are used to model the timber components (Solid X-lam floors and wall panels). The behavior of timber connection is check by considering the stiffness degradation, pinching and post-peak strength during the calibration of experimental cyclic tests on each degree of freedom. The friction effect at the interface between the foundation and panel can also be noticed. We implement springs in experimental model using the software package Abaqus as in external subroutines the springs are used. The 2D schematic diagram of X-lam wall with elastic and isotropic four-node elements is shown in figure 3 which is made in Abaqus software using the poison ratio of 0.35 and young's modulus of 5682 MPa. The different experimental cyclic tests and numerical tests on single X-lam walls, coupled X-lam walls and a single-storey X-lam building are compared and it is conclude that the strength and stiffness degradation parameters were calculated by minimizing the total energy difference between experimental and numerical results. The seismic performance of X-lam buildings and subassemblies can be used to model for seismic response because of their proposed model is very robust.

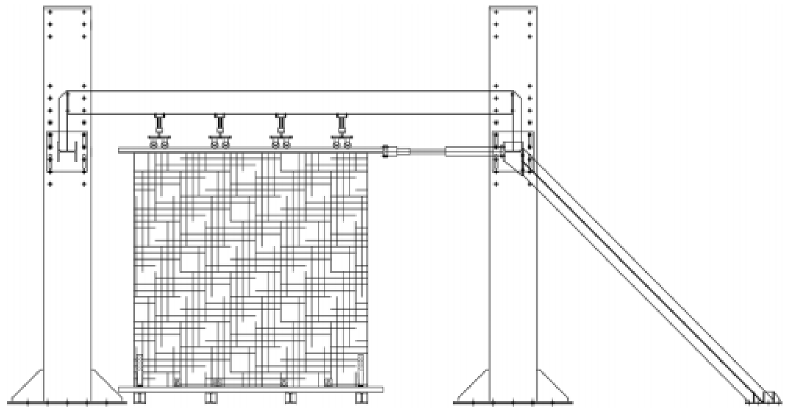

Figure 3. 2D schematic diagram of X-lam wall (Rinaldin et al. 2013)

Table 1: Different types of uplifts used in FEM

Authors	Uplifts	Response	Software's
Khan et al. (2014)	dampers	Used to enhance the performance of structure as they can dissipate more energy during earthquakes.	FEAP SAP2000
Mori et al. (2008)	gap element	Used to minimize damages with respect to fixed foundation	SAP2000
Grange et al. 2009	Macro-element	A macro-element is introduced to simulate the effect of SSI. the global forces and the damage can be reduced by isolate the structure using SSI	FEDEASLab, a finite element MATLAB toolbox
Vassiliou et al. 2016	Rocking frame structure	Uplift at the base and sustain rocking motion are characterized by small residual displacements and small forces transmitted to foundations.	FEM
Rinaldin et al. 2013	multi-spring elements	By lumping a group of springs in an equivalent one, the model can be used for structural design.	Abaqus

4. Reliability of Obtained Behavior

Finite Element modelling is the advanced approach for analysis of structure but totally relying on FEM is not correct. For the checking FEM results must match the FEM results with actual results data. If they are close enough then it is proof our simulation of structure through FEM is correct but if there results are too far from each other then it must be to investigate error either in experimental testing or FEM modelling.
Realistically, the behavior of complicated real structures are studied by the finite element method (FEM)-based approach. The different structural systems are analyzed by this approach. In finite Element modelling it is easily consider various sources of nonlinearity, difficult geometric arrangements, different materials, boundary or support, and connection conditions, and load path to failure. In formulation the material and geometric nonlinearities need to be considered to study the behavior of real steel frame

structures. Furthermore, the various types and forms of connections are used to connect to each other structural members. Fully restrained (FR) connections are usually modeled for steel structures. However, they are partially restrained (PR) or semi-rigid for extensive experimental studies. Thus, to study the behavior of steel frames, it is warranted to consider the realistic rigidity of connections. Major source of nonlinearity in the formulation adds another by the connection rigidity. Mostly the weakness of ordinary steel frames are occur due to non-transfer horizontal loads, e.g., strong earthquakes, high winds, and ocean waves, etc., effectively because of their relative flexibility. The lateral stiffness of flexible steel frames can be increased by the use of reinforced concrete (RC) shear walls. The finite elements is an efficient and logical approach is used to deterministic study of the nonlinear behavior of real structural systems. However, most of the parameters required are random or uncertain in nature for the deterministic evaluation.

5. Conclusion

This paper is written by the literature review of different research papers about uplifts usage and role of uplifts using in FEM by different elements. It is identified that uplift is a phenomena to dissipate maximum energy in structures and minimize the losses of structures during strong ground motions. Using FEM techniques a new trend, it is proved that different types of uplifts are given in vertical elements of structures to resist the lateral forces to minimize losses in which one is gap element which dissipate maximum energy with respect to other types of uplifts. In real scenario gap element is provided in steel structures by bolt which allow the space for the movement of column due to which its collapse duration is elongated to some extent during strong ground motions.
The experimental and numerical results are very close to each other in different types of uplifts and their usage in Software’s which gives the robust results and show that research is not going wrong. Now in current time, further research in this field is going on.

6. Acknowledgement

The authors would like to thank every person/department who helped thorough out the research work, Particularly Mr. Hassam-ud-din and Osma-bin-rizwan for their coordination in this research paper and the careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.”

7. References

Antonellis, G., Gavras, A. G., Panagiotou, M., Kutter, B. L., Guerrini, G., Sander, A. C., & Fox, P. J. (2015). Shake table test of large-scale bridge columns supported on rocking shallow foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, *141*(5), 04015009.

Cui, Y., Lu, X., & Jiang, C. (2017). Experimental investigation of tri-axial self-centering reinforced concrete frame structures through shaking table tests. *Engineering Structures*, *132*, 684-694.

Gregory MacRae, G. Charles Clifton, Michel Bruneau, Amit Kanvinde and Sean Gardiner , “LESSONS FROM STEEL STRUCTURES IN CHRISTCHURCH EARTHQUAKES”, *8th International Conference on Behavior of Steel Structures in Seismic Areas Shanghai, China, July 1-3, 2015*

Jiro Takagi, Akira Wada, “Recent earthquakes and the need for a new philosophy for earthquake-resistant design”, *Soil Dynamics and Earthquake Engineering, Volume 119, 2019, Pages 499-507, ISSN 0267-7261,*

Khan, W., Akhtar, S., & Hussain, A. (2014). Non-linear time history analysis of tall structure for seismic load using damper. *International Journal of Scientific and Research Publications*, *4*(4), 1-5.

Sarrafzadeh, Lim, Qin & Chouw, Effect of plastic hinge, soil nonlinearity and uplift on earthquake energy in structures, Department of Civil and Environmental Engineering, the University of Auckland, Auckland, New Zealand, 2014 NZSEE Conference

Mori, K., Murakami, K., Sakashita, M., Kono, S., & Tanaka, H. (2008). Seismic performance of multi-story shearwall with an adjacent frame considering uplift of foundation. *A a. World Conference on Earthquake Engineering*, Beijing, China

Qin & Chouw (2010). "Experimental investigation of uplift effect on structures in earthquakes." *New Zealand society for earthquake engineering conference, Paper* (Vol. 14).

Qin, Chen, & Chouw (2013). Effect of Uplift and Soil Nonlinearity on Plastic Hinge Development and Induced Vibrations in Structures. Advances in Structural Engineering, 16(1), 135–147.

Rinaldin, G., Amadio, C. and Fragiacomo, M. (2013), A component approach for the hysteretic behaviour of connections in cross-laminated wooden structures. Earthquake Engng Struct. Dyn., 42: 2023-2042.

Stéphane Grange, Panagiotis Kotronis, Jacky Mazars, A macro-element to simulate dynamic, Soil-Structure Interaction, Engineering Structures, Volume 31, Issue 12, 2009, Pages 3034 3046, ISSN 0141-0296,

Vassiliou, M. F., Mackie, K. R., and Stojadinović, B. (2017) A finite element model for seismic response analysis of deformable rocking frames. Earthquake Engng Struct. Dyn., 46: 447– 466.

Wiebe, Lydell and Christopoulos, Constantin and Tremblay, Robert and Leclerc, Martin. "Mechanisms to limit higher mode effects in a controlled rocking steel frame. 1: Concept, modelling, and low-amplitude shake table testing".Earthquake Engineering & Structural Dynamics.

11th International Civil Engineering Conference (ICEC-2020)
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March 13-14, 2020, Karachi, Pakistan.

Effect of Rice Husk Ash on Properties of Self Compacting Concrete Containing Marble Powder

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Abstract

Disposal of waste material is a big environmental issue in Pakistan. Solid waste materials like Marble Powder (MP) and Rice Husk Ash (RHA), have properties to be used in construction as replacement to various ingredients of concrete, hence, avoiding production of huge amount of carbon di oxide associated with manufacturing of cement. In Pakistan, MP is being produced about 300 million tons annually in marble industries and Rice Husk is produced about 22% of total weight of obtained rice crops. These wastes cause adverse effect on environment. To overcome environmental crises, these wastes are being used in construction field to improve the cost and properties of Self Compacting Concrete (SCC). An experimental investigation has been carried out on MP concrete. In this study MP concrete (reference mix) is produced by adding MP as 15% content of OPC while RHA varies up to 25% as cement replacement material. Fresh properties of SCC were measured in terms of Slump flow, L-Box, V-Funnel test, whereas, mechanical properties were investigated in terms of compressive and flexural strength. It is noted that workability of SCC decreased as compared with reference mix by adding RHA. However, hardened properties of SCC increased up to 15% replacement of RHA.

Key Words

Self-Compacting Concrete (SCC), Marble Powder (MP), Rice Husk Ash (RHA), Deterioration, Fresh and Hardened Properties.

1. Introduction

During recent past years Self-Compacting Concrete (SCC) became the most innovative concrete and a part of discussion. Difference between the SCC and Ordinary Concrete is just because of the mixing proportions of materials. SCC known as innovative concrete of the era and having property of self-settlement in construction area without any vibratory force. In other words, no exterior force (Vibrating force) require for placing or settlement of SCC. SCC settles under its own weight by making its path like fluid (Shi et al., 2015). Workability of SCC is more than Ordinary Concrete and also more durable and strengthen then OC (Beygi et al., 2013). SCC is known as innovative because SCC can easily be used at difficult construction sites, like in Composite mesh of reinforcement where vibrator cannot be used. SCC also used in thin places where access of a human being is prohibited or not easy to handle, like in situ piles casting. In simple means, SCC is widely used in congested areas where concreting is not easy. In SCC, noise pollution reduces and improves the filling capability. In Comparison of SCC and OC, SCC can easily pumped to higher elevations easily due to its flow able nature and also enhances the construction speed (Nikbin et al., 2014).

Rice Husk Ash (RHA) is a waste material causes different types of environmental problems and adverse effects on the living organisms (Khan et al., 2012). Rice husk is an outer covering of rice grains produced after milling process. It constitutes 20% of total 700 million tons of Rice crop in the world. In Pakistan, Rice Husk produced about 22% of total weight of Rice crops (Memon et al., 2011). As RHA is a form of ash produced in rice industries after separation of rice with rice husk. Remaining rice husk burn at specific temperature ranges 300^{o}C – 450^{o}C. After that ash produced at controlled temperature 550^{o}C to 700^{o}C for 1 hour and then dump at different disposal sites. This dump causes environmental and health issues. To avoid this type of effects many people tries to use this ash in different purposes. Similarly many researchers use this RHA in different researches to find out unique relationship between RHA and other products (Ameri et al., 2019) (Gill and Siddique, 2018). RHA used as cement replacement has positive effect on SCC (Chopra et al., 2015).

Marble Powder (MP) is also a waste material, produced by the stone cutting or marble industries. MP is not being recycled in marble industries (Topçu et al., 2009). MP just dump outside the industry as waste material. In Pakistan, MP produced about 300 million tons in different marble industries. As this material is used in SCC and gives good performance so this can be used as by product (Alyamaç and Ince, 2009). There are number of studies where individual effect of various MP and RHA was studied. However, limited data is available, even with contradictory results, regarding ternary blend concrete with MP and RHA. Therefore, an experimental investigation has been carried out to evaluate effect of RHA on MP concrete.

2. Experimental Programme

2.1 Materials

2.1.1 Coarse aggregate

This is crushed part of stones and mostly is obtain from mines. Aggregate used during this research is less than 19mm and which meets the requirement of ASTM C33M having properties as given in Table No.1.

2.1.2 Fine aggregate

This material used during experiment having size less than 4.75mm. Fine aggregates which meets the requirements of ASTMC33 were used in this research and having some properties mention in Table No.1.

Table No. 1: Properties of Coarse and fine aggregate

Properties	CA	FA
Surface Texture	Rough	Smooth
Particle shape	Angular	Rounded
Specific Gravity	2.59	2.63

2.1.3 Binder

In this experimental procedure cement named as “Lucky” which is OPC with registration code ISO 9001:2000 and this meets requirement of ASTM C150 having properties mention in Table No. 2.

2.1.4 Rice husk ash

Rice Husk is covering of the rice crops and was obtained from rice fields after milling procedure. Rice husk ash was obtained when rice husk was burnt at controlled temperature as details given in introduction. RHA can be used in Blended form or in unblended form and can make a bond with cementitious material due to its high pozzolanic condition. The RHA used during experimental study as cement replacement and have some properties (which effects the casting and other SCC properties) are mentioned in Table No.3.

Table No.2: Binder Properties

Physical Properties	Results
Normal Consistency %	27
VICAT Initial settling time (min)	150
VICAT Final settling time (min)	245
Specific Gravity	3.073
Le-Chatelier Expansion (mm)	1.68
Compressive Strength at 7^{th} Day (MPA)	24.3
Compressive Strength at 28^{th} Day (MPA)	40.0

2.1.5 Marble powder

Marble powder commonly used during construction. The MP in this experimental research is obtain from mountains or rocks of Taxila, Pakistan. Marble Powder used as filler material during experiment. Some properties of MP are mentioned below in Table No.3.

Table No.3: Properties of Marble Powder and Rice Husk Ash

Physical Properties	MP	RHA
Color	White	Grey
Surface Texture	Smooth	Irregular
Particle Shape	Rounded	Irregular

2.2.6 Water and admixture

Tap water is used for this experimental investigation. Super plasticizer (SP) is a chemical admixture used to enhance the strength of concrete and as water reducing agent in this research. During this research super plasticizer named as Viscocrete-3110 (Sika, 2011) is used to attain high workability. Physical properties of super plasticizer are mention in Table No.4.

Table No.4: Properties of Super-Plasticizer

Physical Properties	Results
Color	Yellowish
Form	Liquid
Density (kg/m^3)	1080
Specific Weight (g/cm^3)	1.17
Chemical	Aqueous solution of modified polycarboxylates, co-polymers

2.2 Mix Proportions

Data of all design mixes given here, where design generates on the basis of self-compacting concrete codes by varying RHA in mixes at 0%, 5%, 10%, 15%, 20% and 25%. While MP remain fixed in each sample about 15% of OPC. These design data is given in Table No.5.

Table No.5: Mix Proportions

Sr. No.	MIX IDs	Cement (kg/m3)	RHA (kg/m3)	MP (kg/m3)	F.A (kg/m3)	C.A (kg/m3)	Water (kg/m3)	SP (% of Binder)
1	RFF	451	0	67.65	655	648	189.6	0.8
2	R5MP	428.5	22.55	67.65	655	648	189.6	0.8
3	R10MP	405.95	45.1	67.65	655	648	189.6	0.8
4	R15MP	383.4	67.65	67.65	655	648	189.6	0.8
5	R20MP	360.85	90.2	67.65	655	648	189.6	0.8
6	R25MP	338.3	112.75	67.65	655	648	189.6	0.8

2.3 Preparation and Testing of Samples

Cubes of size 100×100×100mm (4×4×4in.) are casted for Compressive Strength testing and similarly, beams of Size 100×100×500mm (4×4×20in.) are casted for the Flexural Strength Testing. Cubes and beams cast in respected molds, after demolding samples were placed in tap water for curing purpose.

2.4 Tests on Concrete

2.4.1 Fresh concrete

On fresh concrete slum flow, V-funnel and L-box Test are formed to investigate the filling and passing ability of MP SSC.

2.4.2 Hardened concrete

After completion of curing procedure, Compressive Strength and Flexural Strength Tests are performed on Cubes and Beams respectively. To obtain results three specimens of each SCC mix were tested.

3. Results and Discussion

3.1 Fresh properties

3.1.1 Slump-flow test

Slump flow test used to check the workability of self-compacting concrete. Slump Flow results lies in the range between 650mm to 800mm (EFNARC). Slump flow results obtain after experiments are given in Figure No.1.

3.1.2 V-funnel test

V-Funnel is second workability test perform on fresh concrete to check the flow ability of SCC. The results obtain after V-funnel test lies within acceptable range between 6-12 sec (EFNARC) and results are shown is Figure No.2.

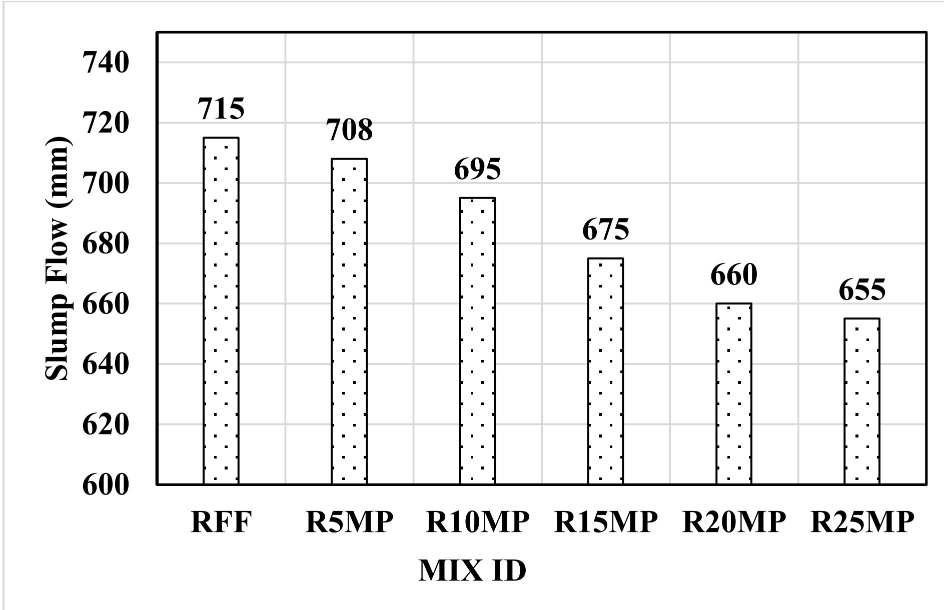

Figure 1: Slump flow results of mixes.

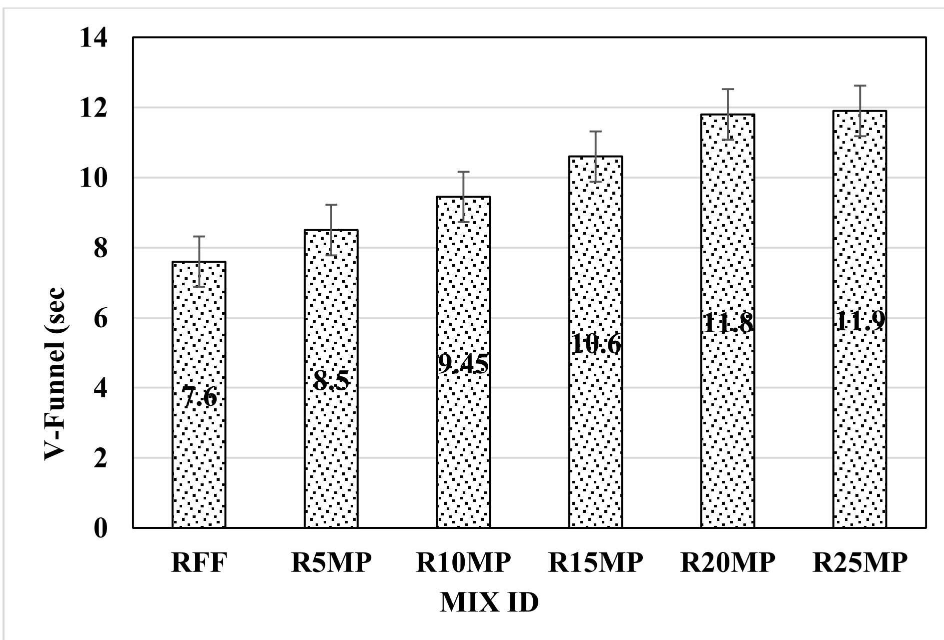

Figure 2: V Funnel results of mixes.

3.1.3 L-box test

L-Box is the third workability test of self-compacting concrete. From L box test, two types of results are obtain. L box test performed to check the passing ability or blockage ratio (h_2/h_1) of SSC.
Range of blockage ratio is 0.8 to 1 (EFNARC). Results obtain from L-Box test, which is within acceptable range. Results are shown in Figure No.3.

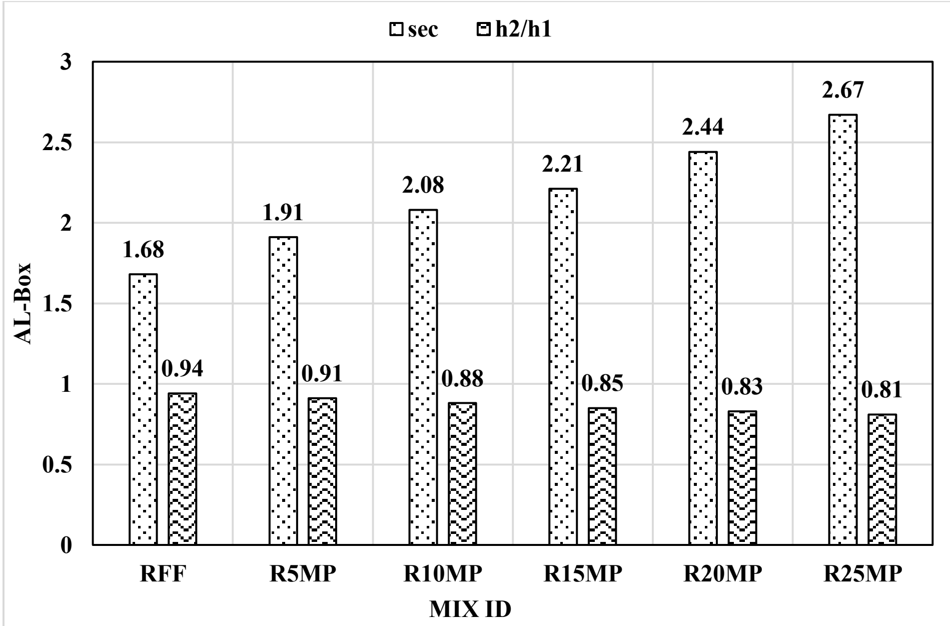

Figure 3: L Box results of mixes.

3.2 Hardened Properties

3.2.1 Compressive strength test

Compressive strength test applied on the hardened concrete samples to check the strength in compression. This test applied after 7, 28 and 91 days to get the results, shown in Figure No.4.

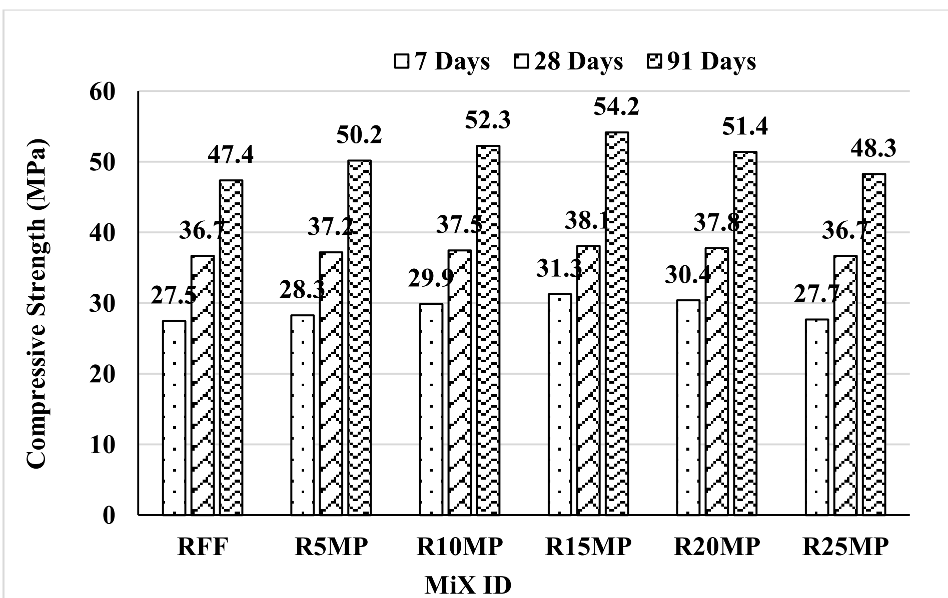

Figure 4: Results of compression testing of mixes.

Further some calculations are made to get the relative analysis of compressive strengths on 7, 28 and 91 days. Which are given in Table No.6. As shown, compression test value increases around 20% at 91days, when 15% RHA used as cement replacement. But after adding more than 15% RHA compression test value start decreasing w.r.t reference sample.

Table No.6: Relative Analysis of Compressive Strength

MIX IDs	$\left[\frac{fc'Mixes-fc'RFF}{fc'RFF}\right]*100$			$\left(\frac{fc'91}{fc'28}\right)*100$
	7days	28days	91days	
RFF	0.0	0.0	0.0	129.2
R5MP	2.9	1.4	5.9	134.9
R10MP	8.7	2.2	10.3	139.5
R15MP	13.8	3.8	14.3	142.3
R20MP	10.5	3.0	8.4	136.0
R25MP	0.7	0.0	1.9	131.6

3.2.2 Flexural strength test

Flexural Strength test used to check the resisting ability of concrete failure in bending. By applying Flexural strength test on SCC mixes sample conducted using one point loading (ASTM C293). Flexure strength results shown in Figure No.5.

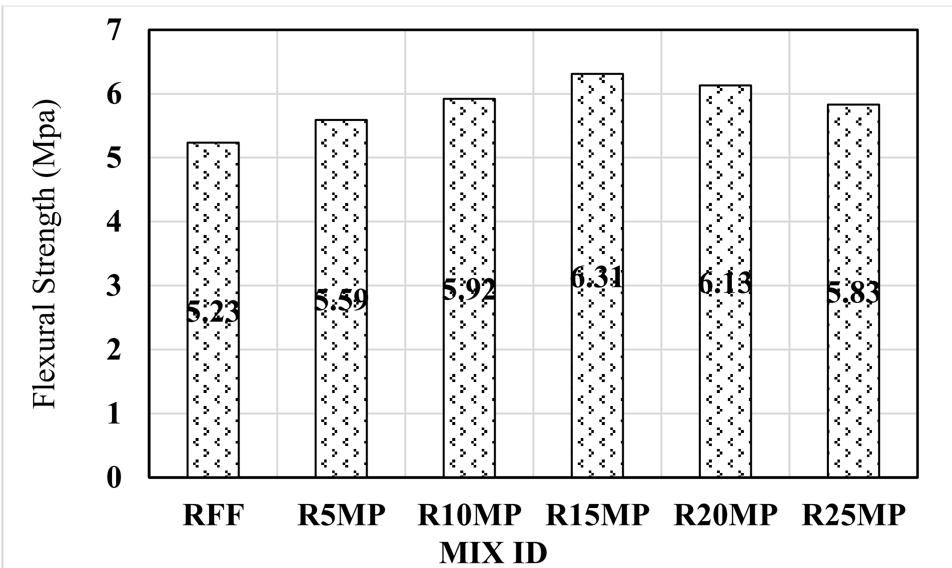

Figure 5: Flexure strength results of mixes.

4. Conclusions and Discussion

This research study investigates the influence of RHA as cement replacement on various parameters including workability, compression strength and flexure strength of Marble Powdered self-compacting Concrete including fresh and hardened properties. After experimental investigation, following conclusions have been drawn:

- It is proved that SCC can successfully be made with fixed content of marble powder and RHA up to 25% of OPC replacement as per EFNARC guidelines.
- Slump flow of SCC mixes decreased with increasing the content of RHA.
- There is also an increase in V-funnel and L-Box results with the addition of RHA.
- Flexural Strength and Compressive Strength of SCC Mixes increased up to the addition 15% RHA. However, a drop in these properties was observed beyond this limit of RHA.

- Mechanical Properties were considerably increased after 28 days due to the pozzolanic characteristics of RHA.

5. References

ALYAMAÇ, K. E. & INCE, R. 2009. A preliminary concrete mix design for SCC with marble powders. *Construction and Building Materials,* 23**,** 1201-1210.

AMERI, F., SHOAEI, P., BAHRAMI, N., VAEZI, M. & OZBAKKALOGLU, T. 2019. Optimum rice husk ash content and bacterial concentration in self-compacting concrete. *Construction and Building Materials,* 222**,** 796-813.

BEYGI, M. H. A., KAZEMI, M. T., NIKBIN, I. M. & AMIRI, J. V. 2013. The effect of water to cement ratio on fracture parameters and brittleness of self-compacting concrete. *Materials & Design,* 50**,** 267-276.

CHOPRA, D., SIDDIQUE, R. & KUNAL 2015. Strength, permeability and microstructure of self-compacting concrete containing rice husk ash. *Biosystems Engineering,* 130**,** 72-80.

GILL, A. S. & SIDDIQUE, R. 2018. Durability properties of self-compacting concrete incorporating metakaolin and rice husk ash. *Construction and Building Materials,* 176**,** 323-332.

KHAN, R., JABBAR, A., AHMAD, I., KHAN, W., KHAN, A. N. & MIRZA, J. 2012. Reduction in environmental problems using rice-husk ash in concrete. *Construction and Building Materials,* 30**,** 360-365.

MEMON, S. A., SHAIKH, M. A. & AKBAR, H. 2011. Utilization of Rice Husk Ash as viscosity modifying agent in Self Compacting Concrete. *Construction and Building Materials,* 25**,** 1044-1048.

NIKBIN, I. M., BEYGI, M. H. A., KAZEMI, M. T., VASEGHI AMIRI, J., RABBANIFAR, S., RAHMANI, E. & RAHIMI, S. 2014. A comprehensive investigation into the effect of water to cement ratio and powder content on mechanical properties of self-compacting concrete. *Construction and Building Materials,* 57**,** 69-80.

SHI, C., WU, Z., LV, K. & WU, L. 2015. A review on mixture design methods for self-compacting concrete. *Construction and Building Materials,* 84**,** 387-398.

SIKA 2011. Concrete Admixture for High Flow/Self-compacting Concrete. UAE.

TOPÇU, İ. B., BILIR, T. & UYGUNOĞLU, T. 2009. Effect of waste marble dust content as filler on properties of self-compacting concrete. *Construction and Building Materials,* 23**,** 1947-1953.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Effect of Heavy Weight Magnetite Aggregate on Mechanical and Radiation Shielding Properties of Concrete

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Abstract
This paper presents an experimental investigation conducted on mechanical and radiation shielding properties of concrete using heavy weight aggregate (HWA) magnetite as an aggregate, having specific gravity 4.6, in place of normal aggregates. Mechanical properties like compressive strength, splitting tensile strength and modulus of elasticity were investigated using concrete cylinders for 28 days. Mix was designed for the target strength of 45MPa, 38Mpa, 31MPa, 24MPa and 15MPa at 0.33, 0.41, 0.48, 0.57 and 0.68 of water to cement ratio respectively as per ACI 211.1. Samples were tested at the age of 7 and 28 days of curing in accordance with ASTM standards. Radiation shielding ability was studied for concrete subjected to Gamma-rays source Cobalt-60 (CO^{60}). Furthermore, ASTM standard was used to conduct thermogravimetric analysis (TGA) wherein concrete samples were exposed to high temperature. The study concluded that compressive and tensile strengths were increased by a maximum of 18.6% and 17.4% at 28 days and 16% and 14.9% at 7 days of curing respectively. Like ordinary concrete, mechanical properties were degraded as the water to cement ratio increases. Results showed that radiation shielding performance of concrete was enhanced with the incorporation of magnetite aggregate in concrete and 0.41 is recommended as the optimum water to cement ratio with acceptable level of radiation shielding.

Keywords
Normal Concrete, Magnetite Concrete, Compressive Strength, Tensile Strength, Modulus of Elasticity, Radiation Shielding.

1. Introduction
Concrete is one of the most widely recognized materials used for construction all over the world and in Pakistan. It has a very wide use in buildings, dams, roads, hydropower stations, airports, bridges and foundations. Concrete is economical as well as a versatile material. Being a composite material, its properties are variable depending upon the ingredients (Ismail et al., 2012). By substitution of main ingredients or cementitious phase, required properties of concrete are achieved. High density concrete is generally used in structures to create effective barriers against the penetration of different types of harmful rays (Ahmad et al., 2019). And the densification of concrete depends on the its density of ingredients (Ouda 2014). Heavy weight concrete, also called radiation shielding concrete provides protective barrier to absorb X-rays and Gamma-rays as it stops or changes the direction when striking its surface. The density and mechanical properties of heavy weight magnetite concrete were found to be more than their counterpart control concrete samples (Sikora et al., 2015). Increasing the percentage of heavy weight aggregates, the density and compressive strength increases (Ramana et al., 2014; Suresh et al., 2015). The microstructural and mechanical properties of heavy weight concrete incorporating barite mineral as an aggregate have been enhanced (Akkurt et al., 2010). The use of nuclear technology; however, have adverse effects on the living environment since they result in ionization of living cells resulting in death of living cells and endangering human life through cancer. One of the basic requirements related to the use of nuclear reactions is safeguarding against them by limiting exposure of

living organisms to the radiations resulting from nuclear reactions (Elmahroug et al., 2013; Maxwell et al., 2008). The specialized materials must have the capability to stop x-rays and gamma-rays. Denser materials are required to stop harmful rays while lighter materials are required to stop neutron (Elmahroug et al., 2013). Heavy metals can stop x-rays and gamma-rays effectively and are conveniently used to stop hazardous radiations (Nwosu et al., 2015) but they cannot effectively stop neutrons, eventually, causes photo neutron emission when strikes by gamma-rays (Elmahroug et al., 2013).

In Pakistan normal concrete in combination with lead is used in radiation therapy bunkers. Normal concrete is not effective in radiation shielding with normal thickness. Purpose of this research work is to modify concrete using magnetite, to make it an economical and efficient construction material for radiation shielding by increasing its density with walls and roofs of normal thickness.

2. Methodology

2.1 Materials

Cement, coarse aggregate, fine aggregate, magnetite, water and ultra superplast-470 admixture were utilized. Ordinary Portland Cement (OPC) market name Askari cement brand, was used as binding material. Coarse aggregates were obtained from local stocks of material in Margalla, Rawalpindi, Pakistan. Maximum size of the coarse aggregate used in the study was 25mm. According to standard ASTM C33, to have the required gradation, a blend of different sizes was formed. Physical properties of the coarse aggregates and its gradation are given in Table 1 and Figure 1(a) respectively.

Natural sand in saturated surface dry (SSD) condition was used in the study. Fine aggregates were obtained from the same local stock of materials in Margalla, Rawalpindi Pakistan. Organic impurities were removed by washing and the sand was made clean. Physical properties of the fine aggregates and its gradation curve are given in Table 1 and Figure 1(b) respectively. Magnetite used in this research was obtained from local sources in Kabal Swat, Pakistan. Physical properties of magnetite are given in Table 1.

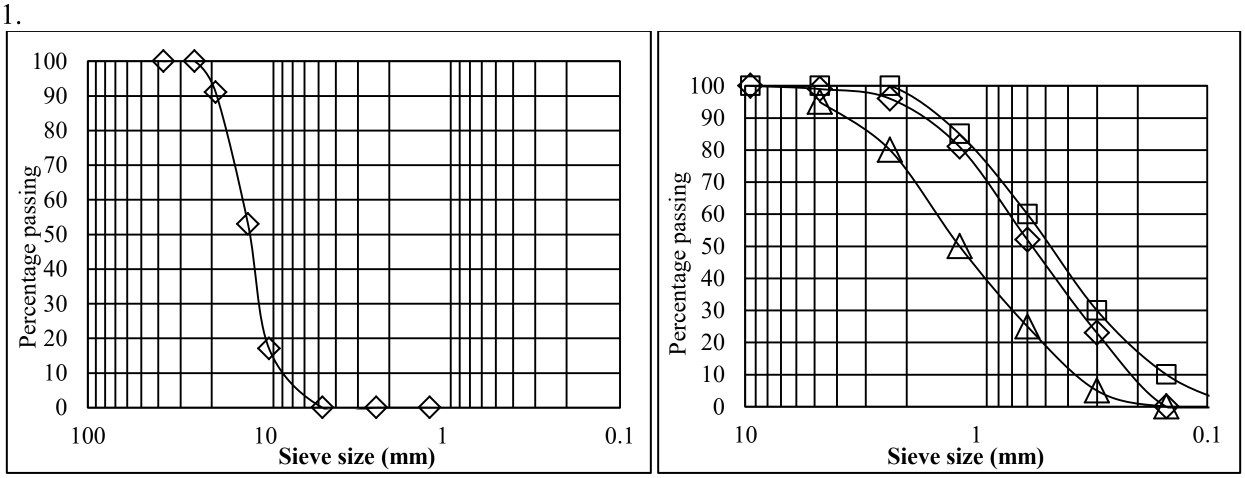

Figure 1 (a) Figure 1 (b)

Figure 1 Gradation curve of coarse aggregates (a) and fine aggregates (b)

Table 1 Physical properties of coarse and fine aggregates

Aggregate	Max aggregate size (mm)	Bulk density (kg/m^3)	Specific gravity	Absorption capacity (%)	Fineness modulus
Coarse	25	1635	2.70	1.30	------
Fine	-------	------	2.40	1.80	2.40
Magnetite	25	1860	4.60	1.40	------

Ultra superplast-470 was used as a chemical admixture to have the required workability of concrete at low water to cement ratio. It is a brown liquid having specific gravity of 1.15 at 20ºC.

2.2 Mix Proportioning

Two separate sets of specimens were produced. First set called normal concrete, consisted of natural coarse aggregates and the second called magnetite concrete with coarse aggregate completely replaced by magnetite. Concrete cylinders of dimensions 150 mm in diameter and 300 mm in height were prepared to conduct the experimental work. Concrete mixes were designed for target strengths of 45MPa, 38Mpa, 31MPa, 24MPa and 15MPa at water to cement ratio of 0.33, 0.41, 0.48, 0.57 and 0.68 respectively. Mix design of concrete was carried out as per ACI method (ACI 211.1) and details are given in Table 2 for normal concrete and Table 3 for magnetite concrete.

Table 2 Mix proportion details of normal concrete

Mix ID	Weight of concrete ingredient (kg/m^3)				W/C Ratio	Chemical admixture (% of weight of OPC)
	Cement content	Fine aggregate content	Coarse aggregate content	Water content		
NC/0.33	539.34	533.42	1094.70	177.98	0.33	1.0
NC/0.41	434.10	613.60	1094.70	177.98	0.41	0.8
NC/0.48	370.79	661.84	1094.70	177.98	0.48	0.5
NC/0.57	312.24	706.44	1094.70	177.98	0.57	0.3
NC/0.68	261.73	744.92	1094.70	177.98	0.68	0

Table 3 Mix proportion details of magnetite concrete

Mix ID	Weight of concrete ingredient (kg/m^3)				W/C Ratio	Chemical admixture (% of weight of OPC)
	Cement content	Fine aggregate content	Coarse aggregate content	Water content		
MC/0.33	539.34	923.67	1319.28	177.98	0.33	1.5
MC/0.41	434.10	1012.20	1319.28	177.98	0.41	1.2
MC/0.48	370.79	1065.46	1319.28	177.98	0.48	0.9
MC/0.57	312.24	1114.71	1319.28	177.98	0.57	0.5
MC/0.68	261.73	1157.20	1319.28	177.98	0.68	0

Due to lower water-cement ratios admixture was used to achieve workable concrete. Slump test results are given in Table 4.

Table 4 Fresh concrete properties

Water/Cement	Slump type	Slump value (mm)	
		Normal concrete	Magnetite concrete
0.33	True	35	25
0.41	True	63	38
0.48	True	66	64
0.57	True	81	69
0.68	True	89	78

2.3 Laboratory Testing

2.3.1 Compressive Strength Test

Compressive strength test was conducted on 7 and 28 days cured samples according to ASTM C31/39. Test was performed on cylindrical concrete specimens as shown in Figure 2(a). Concrete cylinders were prepared in three equal layers and for proper compaction, each layer was rodded 25 times. Keeping samples for 24 hours in molds, they were then demolded and cured for 7 and 28 days in the curing tank.

2.3.2 Tensile Strength Test

Split tensile strength test was conducted on cylindrical concrete specimens after 7 and 28 days of curing to determine the splitting tensile strength of concrete in accordance with ASTM 496C/496M. Test setup is shown in Figure 2(b).

2.3.3 Elastic Modulus Test

Ultrasound pulse velocity method was used to determine the dynamic modulus of elasticity according to ASTM C597-09 by positioning the source and receiver on either side of the sample. Compressional waves are passed through the samples from one side and are measured on the other side by an electronic transducer. Time taken by the wave is measured and using equation 1, modulus of elasticity is measured. Test setup for elastic modulus is shown in Figure 2(c).

$$E = \frac{\rho V^2(1+\nu)(1-2\nu)}{(1-\nu)} \qquad (1)$$

2.3.4 Thermogravimetric Analysis

Thermogravimetric analysis indicates the change in weight of specimen with time, as the temperature rises at a uniform rate and was carried out in accordance with ASTM-E1131. TGA measures the stability of samples at intense temperature. Experimental setup is shown in Figure 2(d).

2.3.5 Gamma-Rays Dosimetry Test

For production of gamma rays, Cobalt-60 (Co^{60}) was used as a source of radiations and the experimental setup is shown in Figure 2(e). Coefficient of the linear attenuation (per cm) of gamma rays for control and magnetite concrete was then computed using equation 2.

$$\mu = \frac{1}{x} ln \frac{N_\circ}{N} \qquad (2)$$


Figure 2 (a)


Figure 2 (b)


Figure 2 (c)


Figure 2 (d)


Figure 2 (e)

Figure 2 Laboratory Testing, Compressive strength test (a), Split tensile strength test (b), UPV test (c), Thermogravimetric analysis (d), Gamma-rays dosimetry test (e)

3. Results

3.1 Compressive Strength Test Results

Compressive strength for normal and magnetite concrete was determined by crushing cylindrical concrete specimens in hydraulic universal testing machine after 28 and 7 days of curing. Results are given in Figure 3 (a) for normal concrete and in Figure 3 (b) for magnetite concrete.

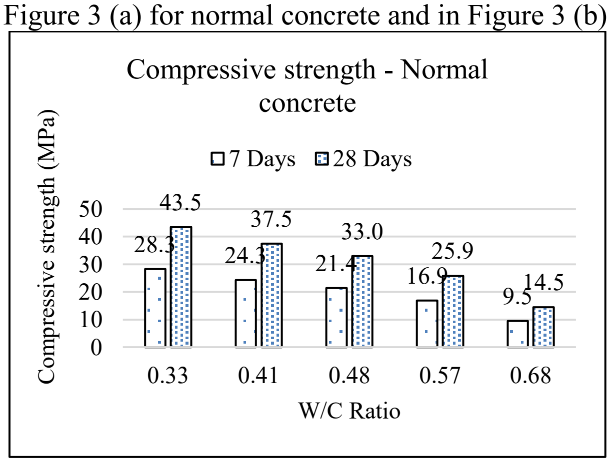

Figure 3 (a)

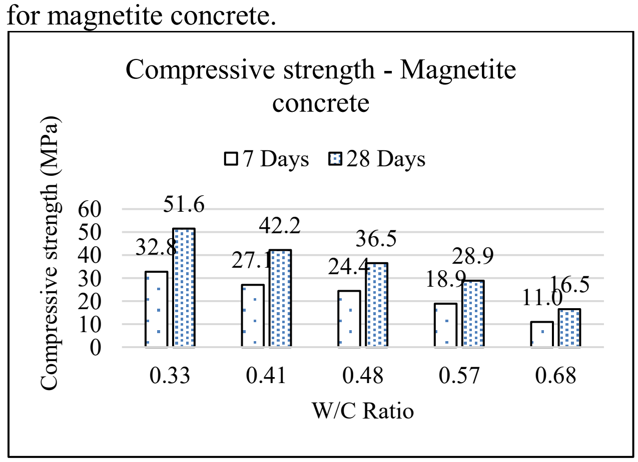

Figure 3 (b)

Figure 3 Compressive strength test results, Normal concrete (a), Magnetite concrete (b)

The rate of gaining compressive strength of normal concrete with time of curing is found to be more, that is 42.5% at lowest water to cement ratio i.e. 0.33 and then 36.5%, 32%, 24.9% and 13.5% at water to cement ratio of 0.41, 0.48, 0.57 and 0.68 respectively.

For magnetite concrete, the rate of gaining compressive strength with time of curing is found to be more, that is 50.6% at lowest water to cement ratio i.e. 0.33 and then 41.3%, 35.5%, 27.9% and 15.5% at water to cement ratio of 0.41, 0.48, 0.57 and 0.68 respectively.

From results it is evident that maximum increment of 18.6% has occurred in compressive strength of magnetite concrete at water to cement ratio of 0.33 at 28 days of moist curing. And maximum increment of 16% has occurred in compressive strength of magnetite concrete at water to cement ratio of 0.33 at 7 days of moist curing.

3.2 Tensile Strength Test Results

Tensile strength was determined by crushing concrete samples in hydraulic universal testing machine after 28 and 7 days of curing. Split tensile test results are given in Figure 4 (a) for normal concrete and in Figure 4 (b) for magnetite concrete.

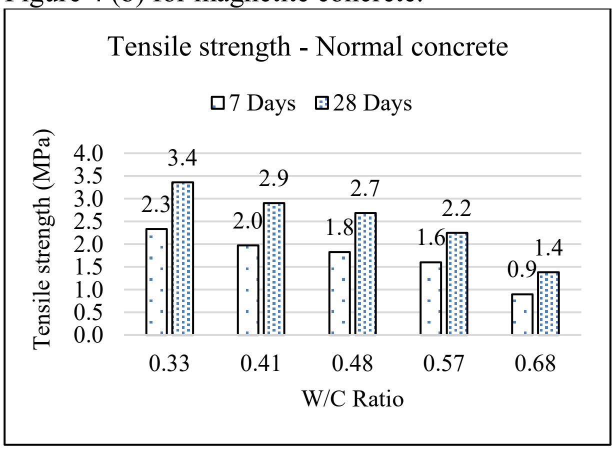

Figure 4 (a)

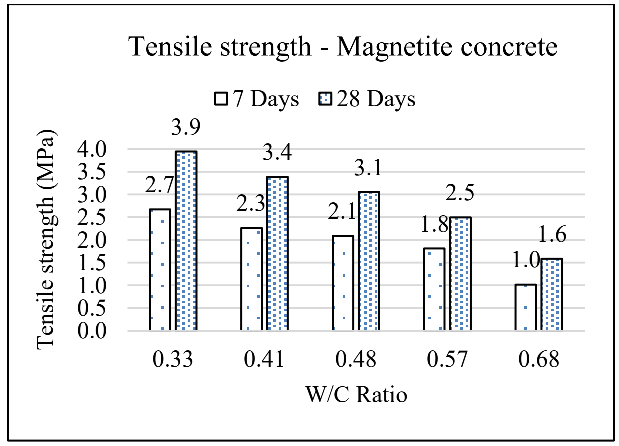

Figure 4 (b)

Figure 4 Tensile strength test results, Normal concrete (a), Magnetite concrete (b)

The rate of gaining tensile strength of normal concrete with time of curing is found to be more, that is 2.4% at lowest water to cement ratio i.e. 0.33 and then 1.9%, 1.7%, 1.3% and 0.4% at water to cement ratio of 0.41, 0.48, 0.57 and 0.68 respectively.

For magnetite concrete, the rate of gaining tensile strength with time of curing is found to be more, that is 2.94% at lowest water to cement ratio i.e. 0.33 and then 2.4%, 2.1%, 1.5% and 0.6% at water to cement ratio of 0.41, 0.48, 0.57 and 0.68 respectively.

From results it is evident that maximum increment of 17.4% has occurred in tensile strength of magnetite concrete at water to cement ratio of 0.33 at 28 days of moist curing. And, exceptionally, maximum increment of 14.6% has occurred in compressive strength of magnetite concrete at water to cement ratio of 0.41 at 7 days of moist curing.

3.3 Elastic Modulus Test Results

Ultrasound pulse velocity test was conducted on concrete samples using Pundit apparatus to determine dynamic elastic modulus after 28 and 7 days of curing. Figure 5 (a) shows test results for elastic modulus at 28 days of curing ad Figure 5 (b) shows results for 7 days of moist curing.

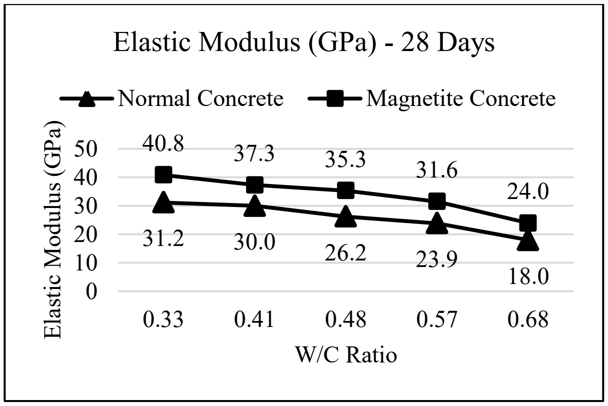

Figure 5 (a)

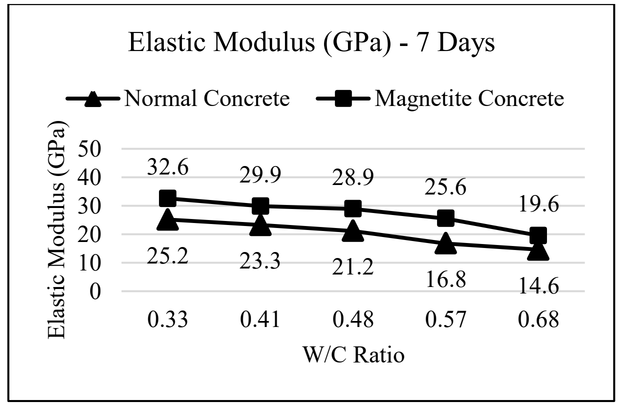

Figure 5 (b)

Figure 5 Elastic modulus test results, 28 days (a), 7 days (b)

From above results it is clear that magnetite aggregate has profound effect on the modulus of elasticity of concrete. 34.7% increase has occurred at water to cement ratio of 0.48 after 28 days of curing. And at the same water to cement ratio 36. 4% increase has occurred after 7 days of curing compared to control concrete samples.

3.4 Thermogravimetric Analysis Results

TGA was performed on concrete samples to check the stability of concrete at intense temperatures. Figure 6 shows that concrete stability has been enhanced at intense higher temperature with the replacement of normal aggregate by magnetite aggregate.

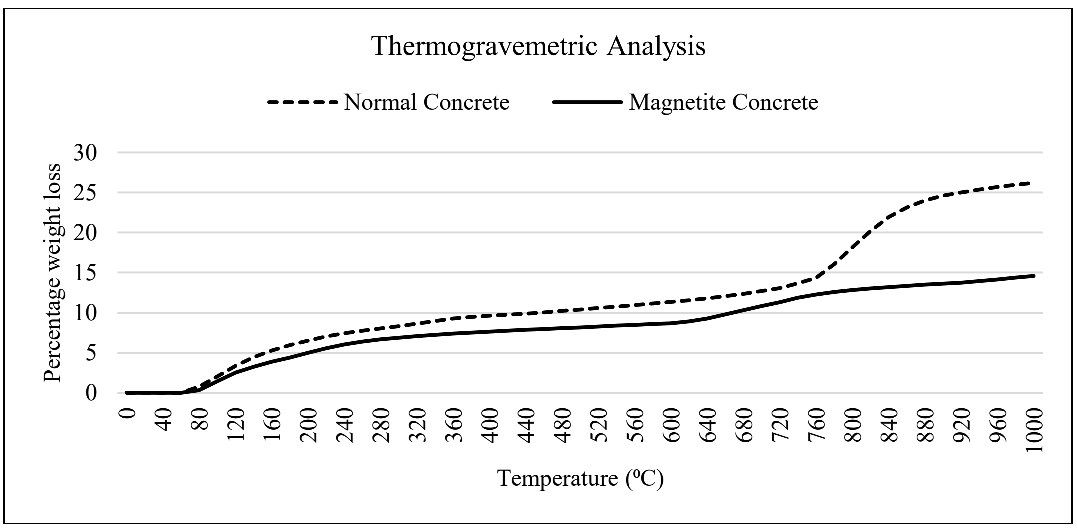

Figure 6 Thermogravimetric analysis results of normal and magnetite concrete

From above results it is evident that in case of control samples abrupt change has occurred at temperature ranges from 650°C to 900°C. After 700°C the percentage difference in weight loss in increasing and maximum difference of 13.6% occurs at 1000°C. There is no significant effect of magnetite aggregates on properties of concrete against heat as normal concrete and magnetite concrete have the same amount of cement.

3.5 Gamma-Rays Dosimetry Test Results

Linear attenuation coefficient is determined for both control and magnetite concrete to compare their shielding ability. And it was concluded that shielding potential of concrete has been increased incorporating magnetite in concrete. Results are shown in Figure 7.

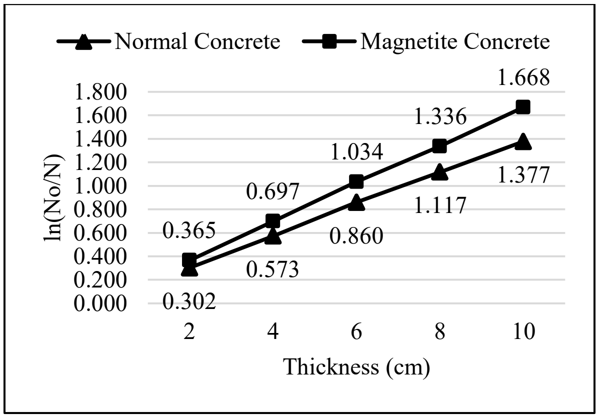

Figure 7 (a)

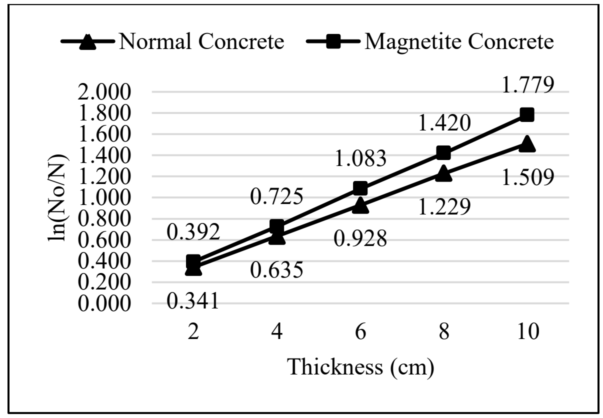

Figure 7 (b)

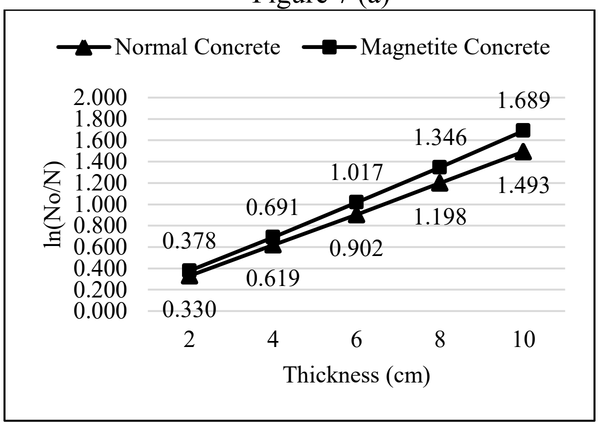

Figure 7 (c)

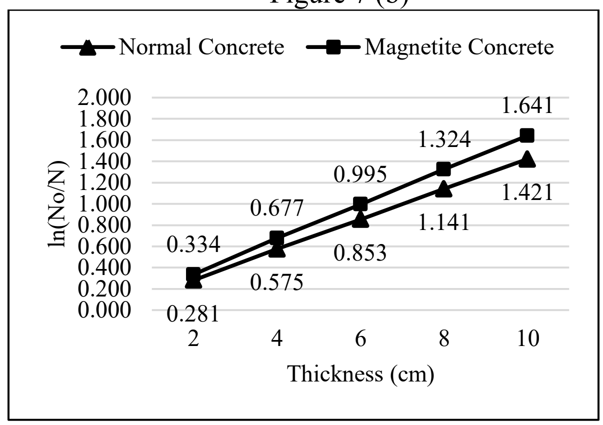

Figure 7 (d)

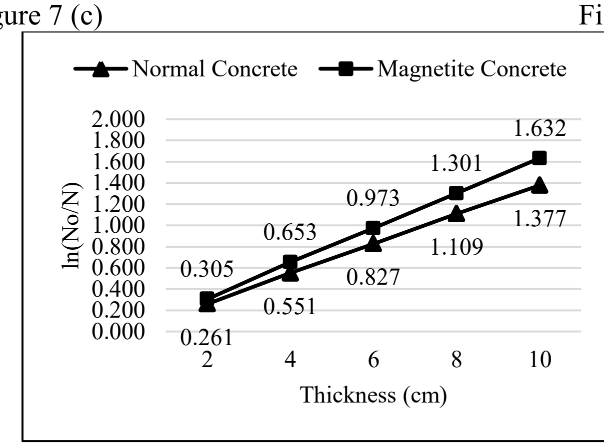

Figure 7 (e)

Figure 7 Gamma-rays dosimetry test results at water to cement ratio of 0.33 (a), 0.41 (b), 0.48 (c), 0.57 (d), 0.68 (e)

From above figure it is concluded that shielding ability of concrete with replacement of magnetite as an aggregate has been improved. As the thickness of concrete samples increases, the linear attenuation coefficient is also increasing. For magnetite concrete samples, linear attenuation coefficient is found to be more than that of normal concrete.

4. Conclusions

Based on the above cited results, the following conclusions may be drawn:

1. Compressive strength and tensile strength were increased by replacing normal aggregate with magnetite aggregates.
2. 8.9% to 18.6% increase in 28 days compressive strength was observed by replacing normal aggregates with magnetite aggregates for all the mixes, exhibiting lowest for MC/0.68 mix and highest for MC/0.33 mix.

3. Tensile strength was observed from 9.6% to 17.4% increase at the age of 28 days by replacing normal aggregates with magnetite aggregates all the mixes, exhibiting lowest for MC/0.68 mix and highest for MC/0.33 mix.
4. Up to 34.7% increase in elastic modulus was observed by replacing normal aggregates with magnetite aggregates.
5. Thermogravimetric results also endorse the trend seen in case of compressive as well as tensile strength within same group of aggregates as rich mixes produced comparatively more weight loss. No significant loss in CH is observed by replacing aggregates.
6. Radiation shielding performance of concrete has been enhanced due to magnetite aggregates. water to cement ratio of 0.41 is observed as optimum water to binder ratio with maximum level of shielding.

5. References

Ahmad I, Shahzada K, Ahmad I, Khan F, Badrashi YI, Ahmad H, Khan SW, Muhammad N. Improvement of concrete shielding to nuclear radiations using barite mineral as fine aggregate. Journal of Engineering Research. 2019 Nov 20;7(4).

Akkurt I, Akyıldırım H, Mavi B, Kilincarslan S, Basyigit C. Photon attenuation coefficients of concrete includes barite in different rate. Annals of nuclear energy. 2010 Jul 1;37(7):910-4.

Elmahroug Y, Tellili B, Souga C. Calculation of fast neutron removal cross-sections for different shielding materials. International Journal of Physics and Research (IJPR). 2013 Jun;3(2):7-16.

Ismail MP, Masenwat NA, Sani S, Muhammad AB, Shah Shamsuddin MK, Rashid RA. Concrete mix design for X-and gamma shielding. InSeminar RND Nuklear (Malaysia,) 2012 Jul 1.

Maxwell CA, Fleisch MC, Costes SV, Erickson AC, Boissière A, Gupta R, Ravani SA, Parvin B, Barcellos-Hoff MH. Targeted and nontargeted effects of ionizing radiation that impact genomic instability. Cancer research. 2008 Oct 15;68(20):8304-11.

Nwosu OB. Comparison of gamma ray shielding strength of lead, aluminium and copper from their experimental and MCNP simulation result. International journal of technology enhancements and emerging engineering research. 2015;3:2347-4289.

Ouda AS. Development of high-performance heavy density concrete using different aggregates for gamma-ray shielding. Progress in Nuclear Energy. 2015 Mar 1;79:48-55.

Ramana KV, Singh BS. Mechanical properties of heavy weight concrete using heavy weight coarse-aggregate as Hematite (Fe58 High Grade Iron Ore). PAN. 2014;15(1.5):100.

Sikora P, Horszczaruk E, Cendrowski K, Mijowska E. The influence of nano-Fe 3 O 4 on the microstructure and mechanical properties of cementitious composites. Nanoscale research letters. 2016 Dec 1;11(1):182.

Suresh A, Abraham R. Experimental study on heavy weight concrete using hematite and laterite as coarse aggregate. International Journal of Engineering Trends and Technology (IJETT). 2015;28(4).

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Effect of Waste Glass Powder on the Fresh and Hardened Properties of Concrete

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Abstract:
In this research work, the waste glass powder (WGP) used as a partial replacement of cement in concrete. The main aim of this research study is to check the workability of fresh concrete, compressive strength, and splitting tensile strength of concrete by using waste glass powder of 0%, 10%, 20% and 30% by the weight of cement. Waste glass is mostly used for landfills as a waste product and to create environmental pollution. To reduce this detrimental effect of waste glass in the environment strategy applied is to use glass waste as a cement replacement material in concrete. In this regard, total 48 concrete specimens were cast with the mix proportion of 1:2:4 with 0.50 water-cement ratio and cured at 7 and 28 days. It was observed that the workability of waste glass powder concrete increased as compared to controlled mix concrete. The result was advocated that the compressive strength of concrete enhanced by 12% and splitting tensile strength improved by 7.5% by using 10% of waste glass powder in concrete after 28 days respectively.

Key Words: Waste Glass Powder, Cement Replacement, Strength of Concrete, Reduce Environmental Pollution.

1. Introduction

Concrete is the most extensively used building resources in the world. Demand for lower energy consumption has augmented due to global warming. The impact of global warming has affected everyone on the planet, which is a universally renowned concept. Cement production requires a high level of energy, which releases a lot of carbon dioxide (CO2) also contributes to greenhouse gas emissions.
Though, the manufacturing of Portland cement leads to the exhaust of huge quantities of carbon dioxide. One ton of Portland cement should generate about 1 to 1.25 tons of carbon dioxide and other greenhouse gases. Over the past 200 years, the range of carbon dioxide has improved by almost 30% in the atmosphere (Yoon et al., 2007; Bheel et al., 2018; Bheel et al., 2019). Environmental issues play an

essential role in the sustainable development of the cement and concrete industry. Part of the cement can be replaced with pozzolanic materials to reduce cement consumption and to some extent check environmental pollution. Every year, about 111 million tons of measured unwanted material from domestic, commercial, and industrial waste is disposed of in landfills in the United Kingdom, resulting in increased disposal costs and environmental problems (Ahmann et al., 2007). Recycling construction debris helps save limited landfill space and waste disposal costs.

The energy required to reuse the recyclable material is less than that of the starting material. The usage of reused ingredients in buildings is the supreme attractive option since a large number of construction sites leads to large amounts of processing. Secondary aggregates can be used instead of asphalt concrete, unbound foundations, pipe laying, landfill gas discharge systems and gravel fillings for drainage systems (Choo et al., 2003). A number of industrial waste, such as ceramic waste, millet husk ash (Bheel et al., 2018), silica fumes, Tile powder (Bheel et al., 2019), fly ash, rice husk ash (Bheel et al., 2018), sugarcane bagasse ash (Dayo et al., 2019), maize cob ash (shaikh et al., 2019) etc., have been identified for use in concrete. Recently, studies have shown that used glass can be effectively applied to concrete in the form of glass aggregate or glass volcanic ash. In the concrete industry, attempts have been prepared to usage spent glass powder as a cementitious material. Numerous studies are underway on the use of Portland cement substitutes using several wastes such as fly ash (PFA), ceramic waste powder, tile powder, and marble dust. For example, PFA and TP, glass powder (GLP) can be utilized as a binding ingredient, partially replacing cement, taking some reactions during hydration, and can also be used as a filler (Shayan et al., 2006; Neithalath, 2011; Schwarz et al., 2008).

When the used glass is crushed into very fine powder, it exhibits pozzolanic properties because it comprises SiO_2. Consequently, it can to certain level substitute cement in concrete, contribute to the growth of strength and increase durability (Chikhalikar S.M. and Tande S.N., 2012). As glass contain huge amount of silica; so, when the particle size is less than 75 microns, it can have a pozzolanic effect (Federico et al., 2009; Jin et al., , 2000).

Owing to increment in disposal prices and ecological alarms, the role of reused glass in Portland cement and concrete has attracted worldwide attention. Glass used for containers, cans, and bottles is a silicate with silicate, which accounts for 80% of the secondary glass (Khatib et al., 2012). The term glass includes several chemicals, including binary alkali metal silicate glass, borosilicate glass, and ternary silicate-calcium-silicate glass. Glass is an amorphous solid that has been found in several forms for millennia and has been produced for people from 12,000 BC. Glass is the most versatile substance on the planet and is used in a variety of fields: from clean glass to tempered and tinted glass.

Due to the emphasis on sustainable buildings, the construction industry is increasingly interested in using scrap or reused constituents in concrete. Glass is an inert substantial that can be repeatedly processed without changing its chemical composition (Anwar, 2016). Research conducted on the usage of broken glass as substitution for aggregates dates back several decades (Anwar, 2016). Although the utilized of thin frosted glass as a pozzolanic resource began in the 1970s (Shayan, 2006), more efforts are done in this region is comparatively new (Shayan A. , 2002; Shao et al., 2000; Shi et al., 2004), and this is stimulated due to the increment of this substance.

In addition to using used glass as cullet for glass production, used glass is also broken into specific sizes used as aggregates for different usage, like water filtration, rough sand plaster, for sports lawns. Sand coating and sand substitution in concrete (Vijayakumar et al., 2013). Due to the growing requirement for concrete production, the use of river sand as a fine aggregate leads to the development of natural resources, lower groundwater levels and lowering of supports. Attempts have been made to use cullet as fine aggregate instead of river sand (Anwar, 2016). Broken glass is too utilized as a large aggregate in manufacturing of concrete; however its flat and thin properties enhance the reduction in workability and lead to a reduction in compressive strength (Christopher, 2011). The main adverse effect with the utilization of waste glass powder in concrete is the chemically reacted among the particles of glass enriched in silica and the alkali in the solution in the pores of the concrete, called the alkaline-silicate reaction (ASR), which can stabilize concrete.

ASR can be mitigated by accumulating a mineral additive to the concrete mix. Usually, mineral impurities are used to reduce ASR, i.e. fly ash, silica fumes, and kaolin. Numerous investigations have demonstrated the capability of these ingredients to inhibit ASR. It is well known that a large amount of glass waste as a filler will reduce the specific gravity of concrete (Mageswari et al., 2010). Actually, glass contains a high amount of silica has led to laboratory investigations of the probability of using it as a raw material for cement production. The split glass powder used as a substitute for cement gave positive results (Batayneh et al., 2007).

Partial replacement of cement by used glass favorably affects the microstructure and stability of the cementing material (Neithalath, 2011). When the used glass is used as a partial substitute for cement, a denser (with fewer pores) and more uniform structure is formed, which is advantageous to moisture absorption, which contributes to the long-term durability of the cement material. By focusing on potentially harmful reactions between cement hydrates and reactive aggregates, the waste glass is used in the place of cement; it also contributes to the stability of the cement material. When mixed-colored glass wastes are crushed to the size of cement particles and used in concrete instead of about 20% cement, this can improve the moisture resistance, durability, and hardened properties of concrete (Shayan A. a., 2006). These improvements occur as a result of beneficial chemical reactions of used glass and cement hydrates, resulting in the formation of chemically stable products that improve the pore system of concrete. The waste glass powder is replaced as cement in concrete, not only increases strength, improves economy, but also increases the durability of concrete (Neithalath, 2011; Schwarz et al., 2008). The experimental study identified the characteristics of concrete containing waste glass powder replaced as cement and compared it with the characteristics of ordinary concrete.

2. Materials

2.1 Cement

In this experimental investigation, ordinary Portland cement was used under the brand name "pakland cement" having consistency; initial setting time and final setting time are 33%, 48 min, and 160 min respectively.

2.2 Aggregates

The Hill sand was used as fine aggregates in research work which passed from #4 sieves and the crushed stone was used as coarse aggregates in experimental work having 20mm in size. These aggregates are locally available in the region of Hyderabad, Sindh. The experimental test outcomes are mentioned in Table 01.

Table 13. Properties of fine and coarse aggregates

Sr. No.	Properties	Fine Aggregates	Coarse Aggregates
01	Fineness Modulus	2.25	--
02	Specific Gravity	2.66	2.62
03	Water absorption	1.40%	0.75%
04	Bulk density (compacted)	122 lb/ft^3	102 lb/ft^3

2.3 Waste Glass Powder

The waste glass was collected from Hyderabad market and turned into glass powder. It is a very solid material. Initially, waste glass was crushed enough to meet the required size and then mixed in concrete as cement replaced material. In this experiment work, a ground waste glass was contained particle size of less than 150 μm and sieved through 75 μm. This material replaces cement with a mixing ratio in concrete.

2.4 Water

Clean and drinking water was used in research work.

3. Research Methodology

This research work was analyzed to check the workability of fresh concrete, compressive strength, and splitting tensile strength of concrete blended with waste glass powder of 0%, 10%, 20% and 30% by the weight of cement. The mold for the cube and cylinder is made of steel with internal dimensions of 100 mm x 100 mm x 100 mm for the cube, and 100 mm x 200 mm for the cylinder. Cement, coarse aggregate and fine aggregate are thoroughly mixed with the mix proportion of 1:2:4 with 0.50 water-cement ratio using a mechanical mixer by following ASTM C192 code procedure. For all samples, place the mold on a table shaker and pour the concrete into the mold in three layers using a tamping bar. The mold remained vibrated for one minute, and all samples remained constant. Cube specimens were tested for the compressive strength and splitting tensile tests were carried out using cylinder specimens on a universal testing machine after 7^{th} and 28^{th} days respectively. Three samples were tested for each batch, and the average intensities of three were indicated as the final result. The mix design for experimental work as summarized in Table 02 (Bheel et al., 2017).

Table 14: Mix Proportion of Research Work

S.No	Mix ID	Cement (%)	Waste Glass Powder (%)	Fin Aggregate (%)	Coarse Aggregate (%)	Water-cement Ratio (%)
01	1:2:4	100	0	100	100	0.5
02	1:2:4	90	10	100	100	0.5
03	1:2:4	80	20	100	100	0.5
04	1:2:4	70	30	100	100	0.5

4. Results and Discussions

4.1 Slump Test

This test leads to measure the workability of fresh concrete in terms of slump reduction by following BS 1881-102:1983 code procedure. The maximum workability of waste glass powder concrete was recorded by 66 mm at 30% WGP and the minimum workability of waste glass powder concrete was recorded by 48 mm at 0% WGP as cement replacement material. The experimental result was showed that the slump value increases with rise in WGP content in concrete as shown in Fig. 01.

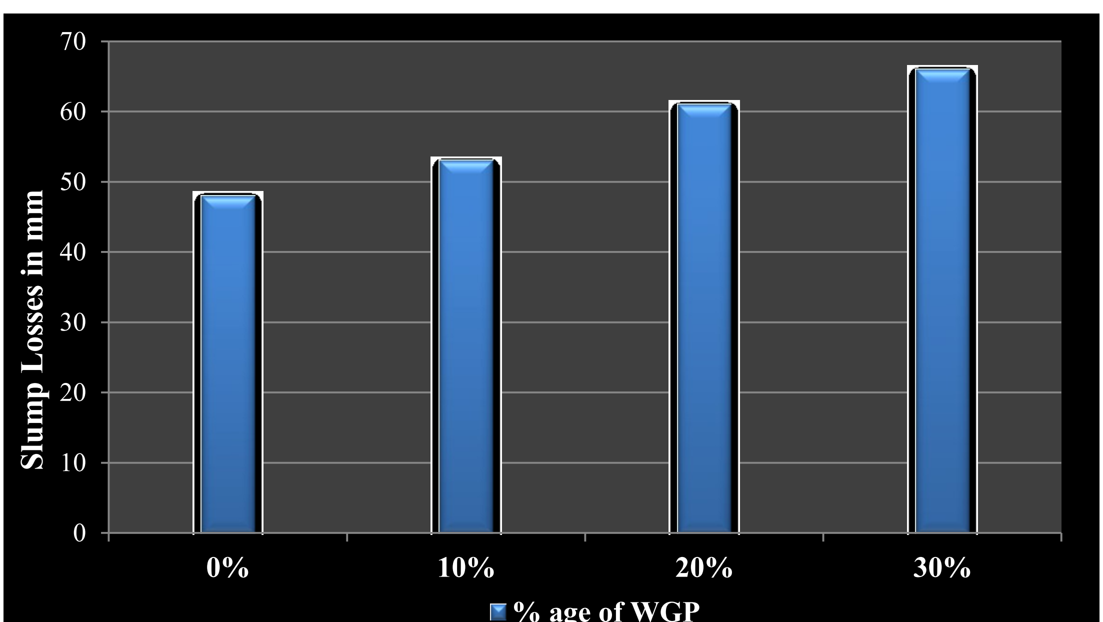

Figure 36: Slump Test

4.2 Compressive Strength

The cube samples were cast for the analysis of compressive strength of concrete blended with different proportions of WGP by obeying BS 12390-3:2009. The highest compressive strength was detected by 22.07 N/mm^2 and 32.13 N/mm^2 at 10% WGP, and lowest compressive strength was measured by 16.70 N/mm^2 and 24.58 N/mm^2 while using 30% of WGP as a cementitious material in concrete for 7th and 28th days respectively. The obtained results are given in Fig. 02.

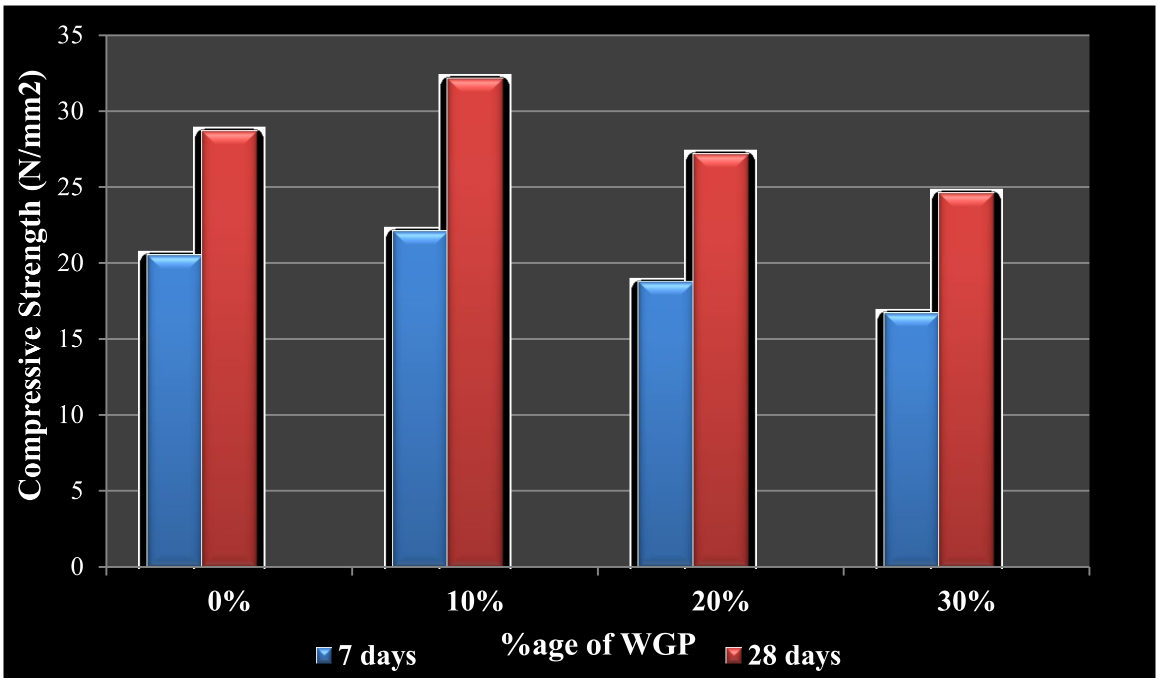

Figure 37: Compressive Strength of Concrete

4.3 Splitting Tensile Strength

The cylinder specimens were analyzed for the determination of splitting tensile strength of concrete by using the different proportion of WGP as per BS 12390-6:2009 code. The maximum splitting tensile strength was detected by 2.30 N/mm^2 and 3.35 N/mm^2 at 10% WGP, and lowest splitting tensile strength was measured by 1.78 N/mm^2 and 2.60 N/mm^2 while using 30% of WGP as cementitious material in concrete for 7th and 28th days respectively. The experimental results are given in Fig. 03.

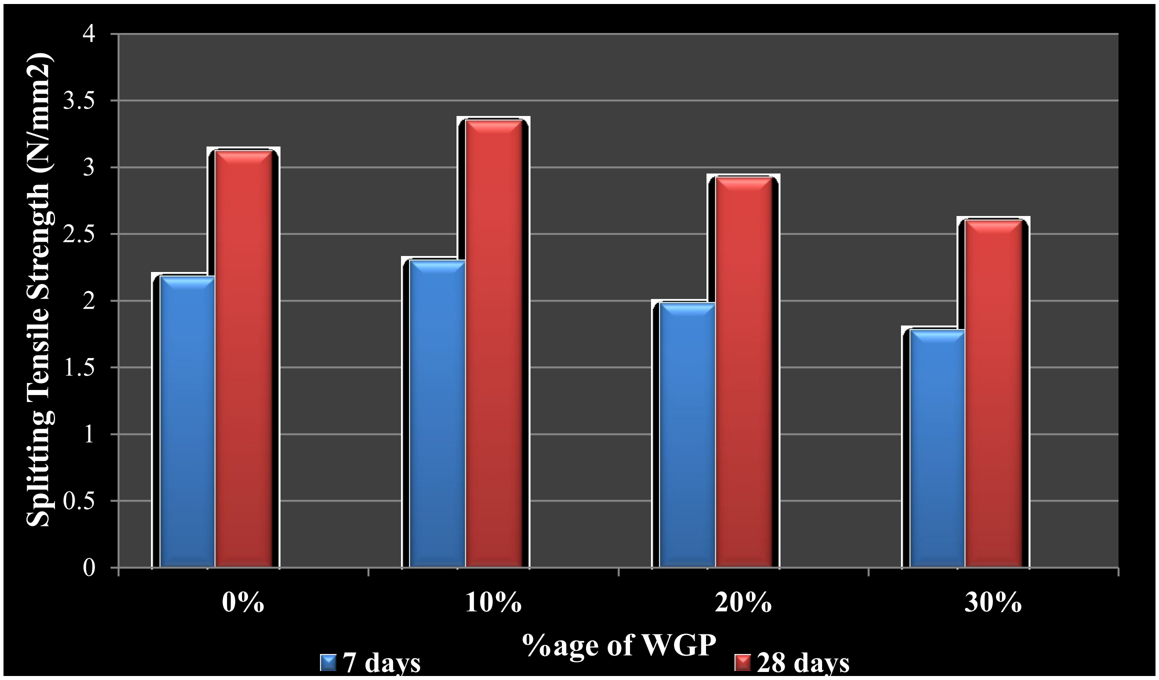

Figure 38: Splitting Tensile Strength of Concrete

5. Conclusions

On the basis of observations and experimental work conducted, the following conclusions were noted down:

- The maximum workability of waste glass powder concrete was recorded by 66 mm at 30% WGP and the minimum workability of waste glass powder concrete was recorded by 48 mm at 0% WGP as cement replacement material.
- The highest compressive strength was detected by 22.07 N/mm^2 and 32.13 N/mm^2 at 10% WGP, and the lowest compressive strength was measured by 16.70 N/mm^2 and 24.58 N/mm^2 while using 30% of WGP as cementitious material in concrete for 7th and 28th days respectively.
- The maximum splitting tensile strength was detected by 2.30 N/mm^2 and 3.35 N/mm^2 at 10% WGP, and lowest splitting tensile strength was measured by 1.78 N/mm^2 and 2.60 N/mm^2 while using 30% of WGP as a cementitious material in concrete for 7th and 28th days respectively.

6. References

Ahmann, D. and Dorgan, J.R.,(2007). Bioengineering for pollution prevention through development of biobased energy and materials state of the science report. Industrial Biotechnology, Vol. 3, No. 3, pp.218-259. https://doi.org/10.1089/ind.2007.3.218

Anwar, A., (2016). The influence of wast glass powder as pozzolanic material in concrete. International Journal of Civil Engineering and Technology (IJCIET), Vol. 7, pp.131-148.

Batayneh, M., Marie, I. and Asi, I., (2007). Use of selected waste materials in concrete mixes. Waste management, Vol. 27, No. 12, pp.1870-1876. https://doi.org/10.1016/j.wasman.2006.07.026

Chikhalikar S.M. and Tande S.N. (2012). An Experimental Investigation On Characteristics Propertiesof Fibre Reinforced Concrete Containing Waste Glass Powder as Pozzolona. 37th Conference on OurWorld in Concrete and Structures, Singapore, August.

Choo, B.S. and Newman, J.B. eds., (2003). Advanced Concrete Technology. Butterworth-Heinemann.

Christopher, C., 2011. Production of sintered light weight aggregate using waste ash and other industrial residues.

Dayo, A.A., Kumar, A., Raja, A., Bheel, N. and Shaikh, Z.H., (2019). Use of Sugarcane Bagasse Ash as a

Fine Aggregate in Cement Concrete. Engineering Science And Technology International Research Journal, Vol.3, No.3, pp. 8-11.
Federico, L.M. and Chidiac, S.E., (2009). Waste glass as a supplementary cementitious material in concrete–Critical review of treatment methods. Cement and concrete composites, Vol. 31, No. 8, pp.606-610. https://doi.org/10.1016/j.cemconcomp.2009.02.001
Jin, W., Meyer, C. and Baxter, S., (2000). Glascrete-Concrete with Glass Aggregate. ACI Materials Journal, Vol. 97, No. 2, pp.208-213. https://doi.org/10.14359/825
Khatib, J.M., Negim, E.M., Sohl, H.S. and Chileshe, N., (2012). Glass powder utilisation in concrete production. European Journal of Applied Sciences, Vol. 4, No. 4, pp.173-176.
Mageswari, M. and Vidivelli, D.B., (2010). The use of sheet glass powder as fine aggregate replacement in concrete. The Open Civil Engineering Journal, Vol. 4, No. 1, pp. 65-71. https://doi.org/10.2174/1874149501004010065
N. Bheel, A.W. Abro, I. A. Shar, A. A. Dayo, S. Shaikh, and Z. H. Shaikh, (2019). Use of Rice Husk Ash as Cementitious Material in Concrete. Engineering, Technology & Applied Science Research, vol. 9, no. 3, p. 4209.
N. Bheel, S.L. Meghwar, S., Sohu, A.R., Khoso, A. Kumar, and Z.H., Shaikh, (2018). Experimental Study on Recycled Concrete Aggregates with Rice Husk Ash as Partial Cement Replacement. Civil Engineering Journal, vol.4, no. 10, pp.2305-2314. https://doi.org/10.28991/cej-03091160
N., Bheel, R.A., Abbasi, S. Sohu, S.A. Abbasi, A.W. Abro, & Z.H. Shaikh, (2019). Effect of Tile Powder Used as a Cementitious Material on the Mechanical Properties of Concrete. Engineering, Technology & Applied Science Research, Volume 9, No. 5, pp: 4596-4599.
N., Bheel, S.L., Meghwar, S.A., Abbasi, L.C., Marwari, J.A., Mugeri, and R.A., Abbasi, (2018). Effect of rice husk ash and water-cement ratio on strength of concrete. Civil Engineering Journal, vol. 4, no. 10, pp.2373-2382. https://doi.org/10.28991/cej-03091166
N.D., Bheel, S.A., Abbasi, S.L., Meghwar, and F.A., Shaikh. Effect of Human Hair as Fibers in Cement Concrete. International Conference on Sustainable Development in Civil Engineering, 23th -25th, November 2017, vol. 01, pp.67-72.
N.D., Bheela, F.A., Memonb, S.L., Meghwar, A.W., Abroa, and I.A., Shara, (2018). Millet Husk Ash as Environmental Friendly Material in Cement Concrete. In Proceedings of the 5th International Conference on Energy, Environment and Sustainable Development, Mehran UET Jamshoro, Sindh, Pakistan: Energy and Environment Engineering Research Group, vol. 1, November 14-16, pp. 153-158.
Neithalath, N., (2011). An overview of the benef its of using glass powder as a partial cement replacement material in concretes. Indian Concrete Journal, Vol. 85, No. 2, pp.9-18.
Schwarz, N., Cam, H. and Neithalath, N., (2008). Influence of a fine glass powder on the durability characteristics of concrete and its comparison to fly ash. Cement and Concrete Composites, Vol. 30, No. 6, pp.486-496. https://doi.org/10.1016/j.cemconcomp.2008.02.001
Shao, Y., Lefort, T., Moras, S. and Rodriguez, D., (2000). Studies on concrete containing ground waste glass. Cement and Concrete Research, Vol. 30, No. 1, pp.91-100. https://doi.org/10.1016/s0008-8846(99)00213-6
Shayan, A. and Xu, A., (2006). Performance of glass powder as a pozzolanic material in concrete: A field trial on concrete slabs. Cement and concrete research, Vol. 36, No. 3, pp.457-468. https://doi.org/10.1016/j.cemconres.2005.12.012
Shayan, A., (2002), January. Value-added utilisation of waste glass in concrete. In IABSE Symposium Report (Vol. 86, No. 6, pp. 12-21). International Association for Bridge and Structural Engineering.
Shi, C., Wu, Y., Shao, Y. and Riefler, C., (2004). AAR expansion of mortar bars containing ground glass powder. Proc. 12th IAARC, Beijing, China, October, pp.789-795.
Vijayakumar, G., Vishaliny, H. and Govindarajulu, D., (2013). Studies on glass powder as partial replacement of cement in concrete production. International Journal of Emerging Technology and Advanced Engineering, Vol. 3, No. 2, pp.153-157.
Yoon, I., O. Copuroglu and K. Park, (2007). Atmospheric Environment, Vol. 41, No. 34, pp. 3-8.
Z.H., Shaikh, A., Kumar, M.A., Kerio, N., Bheel, A.A., Dayo and A.W., Abro. Investigation On Selected Properties Of Concrete Blended With Maize Cob Ash. In proceeding of 10Th International Civil Engineering Conference, NED UET Karachi, 23-24, February, 2019, pp. 23-28.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Energy Dissipation in 3D Structure With Multiple Uplifts at Column Bases

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Abstract

Mitigation measures should be taken against failures of steel or aluminum structures due to earthquakes. A new construction concept has been introduced i.e. allowing uplift in structure to have maximum energy dissipation during strong ground motion. Uplift provision can be made with different mechanisms and one option is loosening of bolt, which could be an effective and cost-efficient solution. Previous studies have focused mainly on the effect of single column uplift (particularly in three-dimensional structure) and favorable results are obtained. In this study, the same 3D prototype structure is considered for studying the effect of multiple uplifts at column bases for possible energy dissipation. The prototype single-storey structure of 55 cm height with a top steel plate of 50 cm x 50 cm x 0.5 cm and with four aluminum columns (having angle section dimensions of 3 cm x 3 cm x 0.3 cm each) is considered. The columns are connected by bolts to a base plate. The prototype structure has been tested by using two different base conditions: i. all four columns are fixed; and ii. two columns are fixed and others two are allowed to have maximum uplifts up to 5 mm. The response of structures and columns uplifts are investigated. Relative comparison has been made between the structural behaviors based on these two conditions. The structure with multiple uplifts has dissipated 57% more energy compared to structure with fixed columns. This is better in relation to structure with single uplift (previous study) which has shown only 24% more energy dissipation. This research has led to understand the significance of the multiple uplifts in 3D structure.

Keywords

Multiple Uplift, 3D Structure, Energy Dissipation, Column

1. Introduction

In seismically active rural areas of developing countries, economic earthquake-resistant structures are desirable. Due to lack of seismic-resistant structures, these areas also suffer substantial loss of life during strong ground motion. Earthquake resistant structures are structures, which are designed to protect buildings to some or greater extent from earthquakes. Over the last ten years, 679,743 persons killed by earthquakes have recorded a large proportion (nearly 59% percent) of the total number of 1,145,015 killed as a result of all-natural disasters. (Reinoso et al., 2017). In Italy, an earthquake came on 6 April 2009 of magnitude 6.3. There are 305 peoples are dead and approximately 1500 peoples were injured in which 134 peoples are died due to improper structure of buildings (Mehmet et al., 2010). According to Bird et al. (2004), the 1999 Chi-Chi, Taiwan earthquake was studied where almost 10,000 building collapse was reported. It was reported that most of the structural damages were caused due to the combined effect of strong ground motions and the design and construction deficiencies. Clifton et al., (2011) examined initial field observations on the performance of selected steel structures damages during the earthquake series of 2010 to 2011 at Christchurch. The majority of the examined structures had minor local damage, but in such structures, severe damages were also observed. Several masonry, concrete and steel buildings have been reportedly damaged in a series of earthquakes in Christchurch between 2010 and 2011. Around 1700 buildings out of 2400 have been destroyed because of tilting or cracking (Takagi and Wada, 2018). New construction techniques have been studied by several researchers during the last decade to enable a cost-effective and efficient solution (Elvin and Uzoegbo, 2011). During the earthquake of Central Italy in 2016, the masonry infill and the partition walls were the most affected. They are mostly caused due to lateral drifts of the structures (Perrone et al., 2019). An earthquake is one of the worst natural disasters that could lead to major damage and human life. Every year there have been more than 3,000 recognizable earthquakes which can result in 10,000 average deaths (Tafsirojjaman et al., 2019). The earthquake in October 2005 was Pakistan's deadliest earthquake. After investigating it was observed that most of the damage was caused by design and implementation deficiencies. There have been more than 400,000 damages to building, approximately 90,000 deaths, 79,000 wounded, 500,000 families were affected and over 3.5 million were homelessness (Maqsood et al., 2008). Hence there is a need to improve the understanding of structural response during an earthquake and incorporate it in design.

Kulkarni et al., (2017) experimentally studied earthquakes by using a shake table. According to the author, shake table plays an important role as a modern tool to assess the behavior of the structure under earthquake as it replicates the ground motion. The structural stability can be assessed by simulating the ground motion during earthquakes efficiently. Cui et al., (2017) conducted experiments using a shake table to investigate the self-centered RC frame structures. Lignos et al., (2013) conducted a full-scale shaking table test of a 4 storey steel moment frame is done to obtain the experimental data for collapse assessment. Nader et al., (1991) investigated experimentally the impact of flexible connections on dynamic structural response. Single-bay-single storey steel structure was built and mounted on a shake table. Energy can be dispersed by shear force-shear displacement and axial force displacement hysteresis behavior. Tomassetti et al., (2019) conducted a shake table test on unreinforced one-storey full-scale masonry structure, which is in addition to a previous test of a two-storey structure with similar characteristics. Climent et al., (2018) conducted a shake table test to investigate the seismic efficiency of waffle-flat plate (WFP) structures on a 2/5-scale test specimen. The lateral strength and stiffness gave in each story by the dampers to test the efficiency of the dampers during the earthquake. Rinaldin et al., (2016) performed a shake table test is on the 3 storey and 7- storey building for the prediction of seismic response. This test gives the results from the non-linear dynamic analysis, so the shaking-table test results are top displacement is within 7.1% and on accelerations within 20%. Now this proposed model can be used in the future to save lives as a European code for seismic design. M. Del Carpio R. et al., (2019) developed 4-storey moment-resisting steel frame building for the evaluation of seismic response through shaking of ground motions. This research mainly shows the experimental response of the gravity framing system through shaking and its contribution to the lateral resistance of a steel frame building. Ceccotti et al., (2013) performed a shake table test on a seven-story structure. For the testing of this prototype, the 3D shaking table was used. The aim was to examine the structure's behavior under harmonic loading.

New techniques for mitigating the impact of earthquakes are being studied. A simple shake table and prototype steel frame structure shall be used to study the dynamic behavior of the structure. There is a new trend to allow uplift in structures to have maximum energy dissipation during strong ground motion. Many researchers are currently working on this new technique. The self-centering method of rocking structural elements has been proposed to reduce seismic damage to steel buildings. The effects of rocking vibrations (uplift response) will minimize seismic damage to buildings subject to strong ground motions (Azuhata et Al., 2008). Huckelbridge et al., (1977) earlier discovered through experimentation that the intentionally allowed uplift in the structures can reduce the design demands as more forces get lost during the ground motion. Baran et al., (2011) study the dynamic behavior of structure by developing simple and low-cost shake tables in the laboratory. The possible advantages of structural uplift in reducing the structural effects of earthquake load have also been studied in past years. The activated forces in structures during an earthquake can be reduced through allowable uplift in structures as a mitigation measure (Qin and Chouw, 2010). Sarrafzadeh et al., (2014) experimentally studied the effect of uplift, plastic hinge, and soil nonlinearity. A single degree of freedom scaled structure was used for the study. Acceleration, displacement and energy dissipation were recorded. Wiebe et al., (2012) proposed a control rocking steel frame built to make it efficient in a way to minimize the structural damages and residential destructions that are due to the seismic force-resisting systems. A low amplitude shake table is used for the analysis of the eight-storey prototype. Wang and Li, (2006) carried out several tests on a semi-rigid full-scale two storey steel frame to investigate the overall response of the structure under the application of vertical load. Okazaki et al., (2011) investigated the Steel building damages that were found following the earthquake in Tohoku-Oki in 2011. Several small and major structural damages have been recorded locally and globally. As was recorded, there was significant damage to a concentrated braced frame structure occupied as an office. In the E-Defense facility in Japan, a series of large-scale dynamic shaking table tests are performed to validate the system behavior for ground motions with intensities equal to the earthquake level. In this research, a controlled rocking steel braced frame system is built. The uplifts are provided at the base of the columns under earthquake loading and minimize the forces in the frame members that built to stay in elastic size (X. Ma et al., 2010).Goggins et al., (2018) developed a test Frame showing a couple of specimen strap-gusted plate and load cell swivel rods used to attach the bottom of the platform. Each test examined two identical brace-gusset platform specimens that are produced in the pair of cold-formed tubes and plates from the same metal. Bolt usage has shown to be effective and can offer a more efficient construction option, rather than welds for connecting the gusset plate flanges to the beam and column members. The maximum base shear in test structure with column uplift was observed to get reduced as compared to that with a fixed base (Midorikawa et al., 2006). In this study, the dynamic behavior of the structure with multiple column uplifts is investigated. Acceleration-time histories, displacement-time histories, base shear- displacement curve and energy dissipation are focused.

2. Experimental Procedures:

2.1 Single-Bay-Single Storey Structure with Multiple Uplift Mechanism

Figure 1 shows the simple 3D single storey prototype frame structure of 55 cm high consists of a top steel plate (50 cm x 50 cm and 0.5 cm thickness) with four aluminum corner columns (each with its dimensions of 3 cm x 3 cm x 0.3 cm each). It has a mass of around 45 kg. The columns are connected by bolts to a base plate. The base plate serves as a link between the structure above and the shake table. Furthermore, only two of four columns bolt were loosened and allowed to have a maximum uplift up to 5 mm. The loosened bolts serve as a guide and as a constraint in conjunction so that it cannot slide off horizontally, except on the vertical axis. Figure 1 shows an enlarged view of the allowed column uplift in the 3D structure.

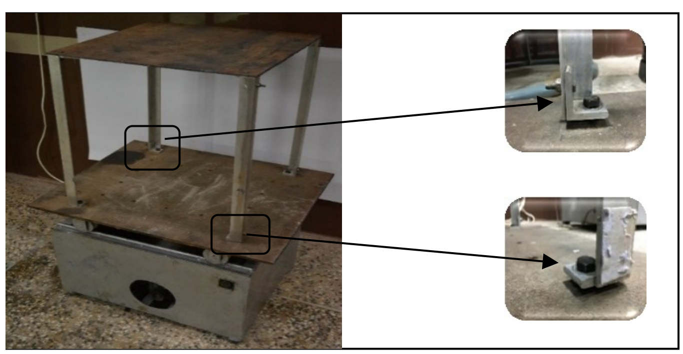
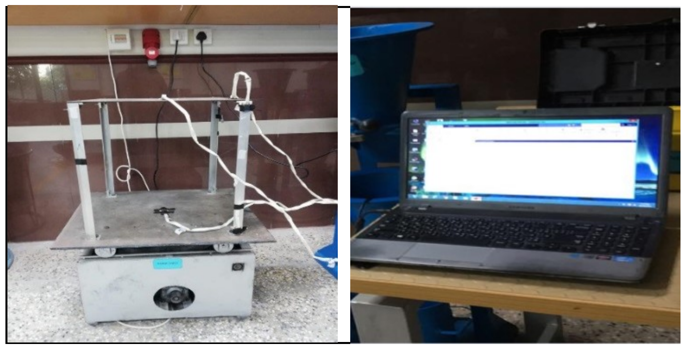


Figure 1: Multiple Column Uplifts in prototype structure

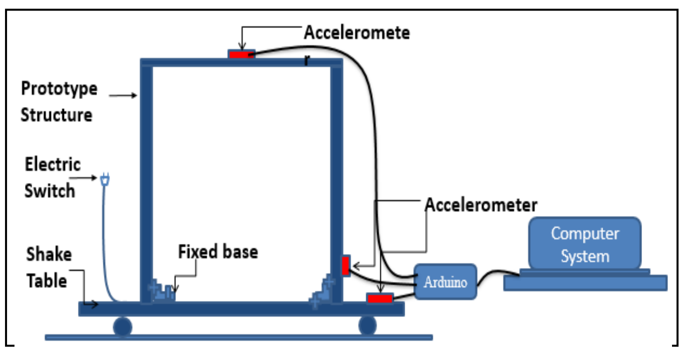
2.2 Shake Table Test and Instrumentation

The prototype structure is tested using two different base conditions i.e. i. all four columns was fixed, and ii. two columns were fixed and two were allowed to have a maximum uplift up to 5 mm. Uplift is allowed in only two diagonal columns through a loosened bolt at the base. The experimental setup for the 3D prototype structure mounted on the shake table along with the details of its instrumentation for response measurement is shown in Figure 2. The structure was instrumented with the following set of accelerometers:

i) one at the shake table to record the applied loading.
ii) one at the bottom of the column with an uplift to record its response i.e. lateral drift.
iii) one at the bottom of another diagonal column to record the uplift.
iv) one at the diaphragm middle to record the structural response against applied loading.



a. Experimental setup



b. Schematic diagram
Figure 2: Shake table testing Setup

2.3 Testing

Experimental testing of the prototype 3D structure is done in the laboratory. The ground motion during an earthquake event can be simulated using a shake table. A simple 1D shake table will be used to apply the harmonic loadings of 1.03 Hz (i.e. rotation of 62 rpm or time period ‘t’ of 0.97 s. As mentioned earlier, the column uplift was allowed by the loosened bolts at the base of the diagonal columns. Due to the payload restriction of the shake table, no extra mass was imposed on top of the structure. This test was conducted solely for evaluating energy dissipation due to allowed structural uplift. Snapback and shake table tests were done under two base conditions with a fixed and allowed uplift at diagonal columns. In order to determine the approximate damping ratios and the fundamental frequency of the 3D prototype structure, a snapback test was conducted.

3. Results

3.1 Acceleration and Displacement-Time Histories

The Structural response as shown in figure 3 is recorded in terms of acceleration and displacement-time histories. As the only parameters analyzed are horizontal acceleration and displacement at the top of the uplifted column and total energy dissipation in structure with a fixed base and allowed uplift. Figure 4 shows the response of the top of the uplifted column during the time period of 15 to 25 seconds in terms of the acceleration-time and displacement time histories. Shake table movement of applied loading is represented by blue full line while the dashed red line shows the response of the structure. The acceleration-time history recorded through accelerometers is processed using a seismosignal program to obtain displacement time histories. As a simple shake table was used to introduce harmonic loading, so the obtained times-histories are the most reliable and appropriate to study the structure's dynamic response.

The structured movement can be divided into three phases under applied loading: a. when the structure started to vibrate until it was in a stable state. b. stable state and c. free vibration of the structure. Figure 4 represents the column uplift provided at the column base during the time period of 10 seconds. When the structure deflected towards the left side, uplift is observed at the right side (i.e. Column-A) and the top relative displacement is to the left side. On the other hand, when the structure deflected towards the right side, uplift is observed at the left side (i.e. Column-A) and the top relative displacement is to the right side.

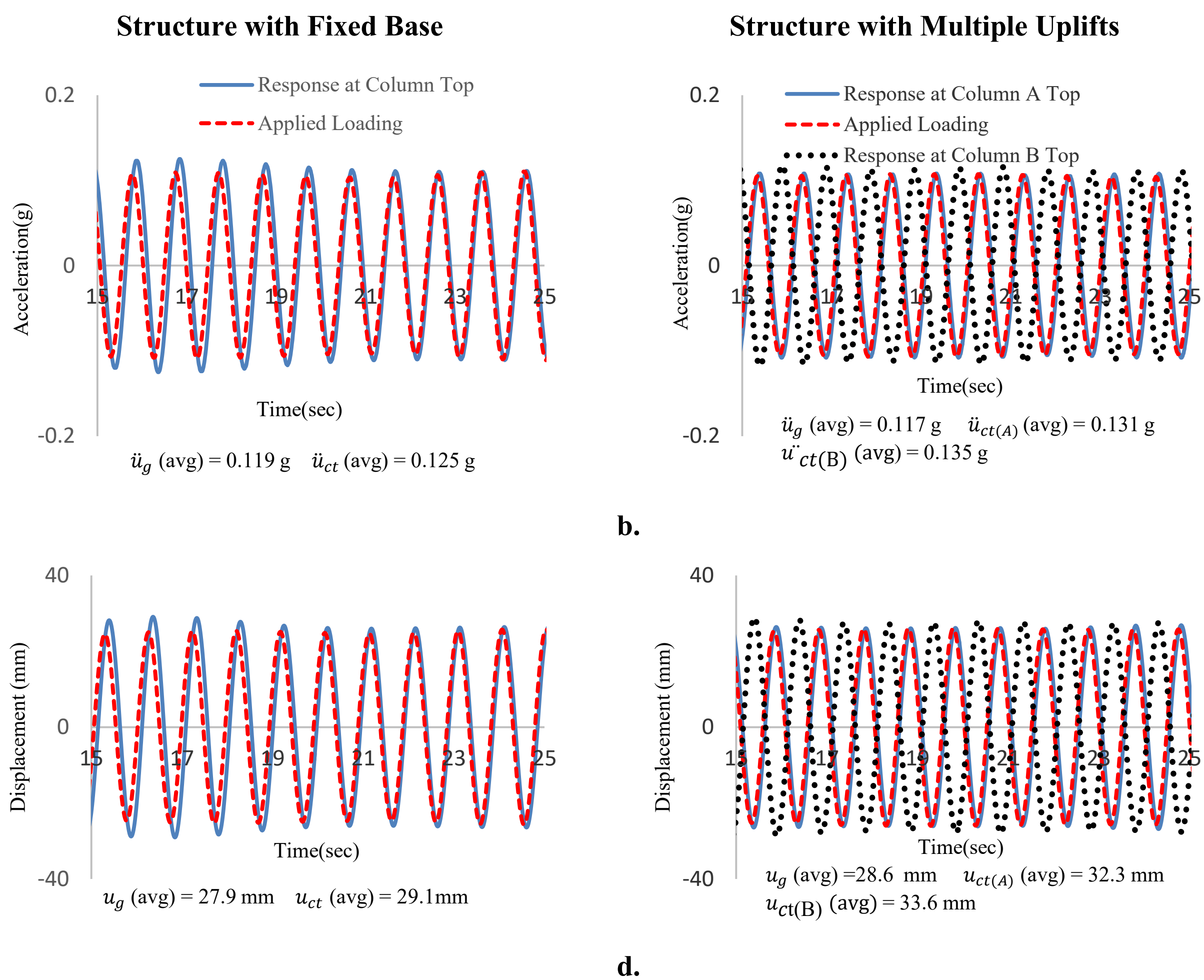


Figure 3: Response of column for intermediate 10 seconds at a frequency of 1.03 Hz
a. acceleration-time history of SWFB, b. acceleration-time history of SWMU, c. displacement-time history of SWFB, and d. displacement-time history of SWMU

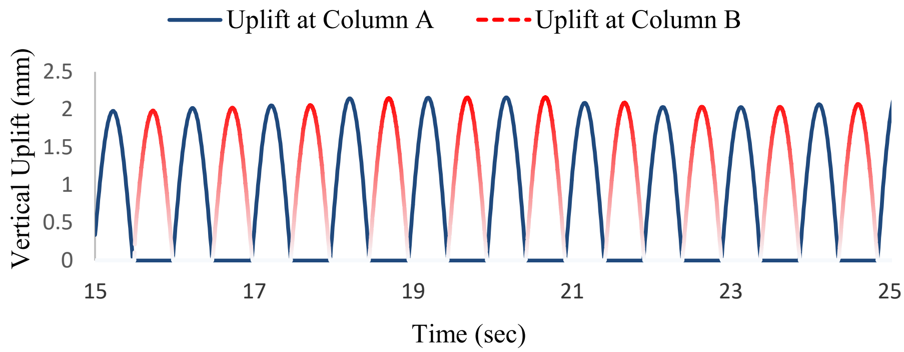


Figure 4: Intermediate vertical Uplift-time history at a frequency of 1.03 Hz

3.2 Base Shear-Displacement (Q-Δ) Curves and Energy Dissipation

The total mass of structure ($\ddot{M}_t$) is assumed to be lumped at the middle of the diaphragm where its response acceleration-time history is recorded (i.e. $\ddot{u}_m$-t). The base shear (Q) is calculated as Σ (M. $\ddot{u}_m$) where the acceleration ($\ddot{u}_m$) is determined at the middle of the diaphragm and $\ddot{M}_t$ is the added mass for ($\ddot{u}_m$). The mean energy dissipation and total energy dissipation within one cycle are shown in Table 1. Figure 5 shows a typical basis shear – displacement curve during complete dynamic loading along with enlarged typical single loop for one cycle. The area within the loop is taken as absorption of energy in one complete cycle. If the area within loop will be increased then energy dissipation will also be increasing. we observed uplifted steel structure at column base connection can dissipate more energy as compared to fixed steel structure, during dynamic loading. Basically, the uplifted columns allow the structure to translate vertically due to loosened bolts at the column bottom. The uplift in the column during an intermediate interval of 10 seconds dynamic loading is shown in figure 4. The uplift occurred in every cycle when the structure shifted opposite the direction of the uplifting column. This uplift at the column-base connection provides flexibility in the structure and due to that, the generated energies get dissipated. we observed relative displacement at the column top because of the movement of the base. we used origin software for a calculating area within the loop to obtain the energy dissipation in structure. The total amount of energy dissipated is significantly increased by allowing the column to be uplifted under harmonic loading in a simple 3D structure.

Table 1 Experimental energy absorption during harmonic loading

Frequency (Hz) for u_g= 30 mm	Total no. of cycles (n)	Energy Absorption			
		Average Energy dissipated in one cycle (J)		Total Energy dissipated (J)	
		SWFB	SWMU	SWFB	SWMU
1.03	62	0.011	0.018	0.706	1.111

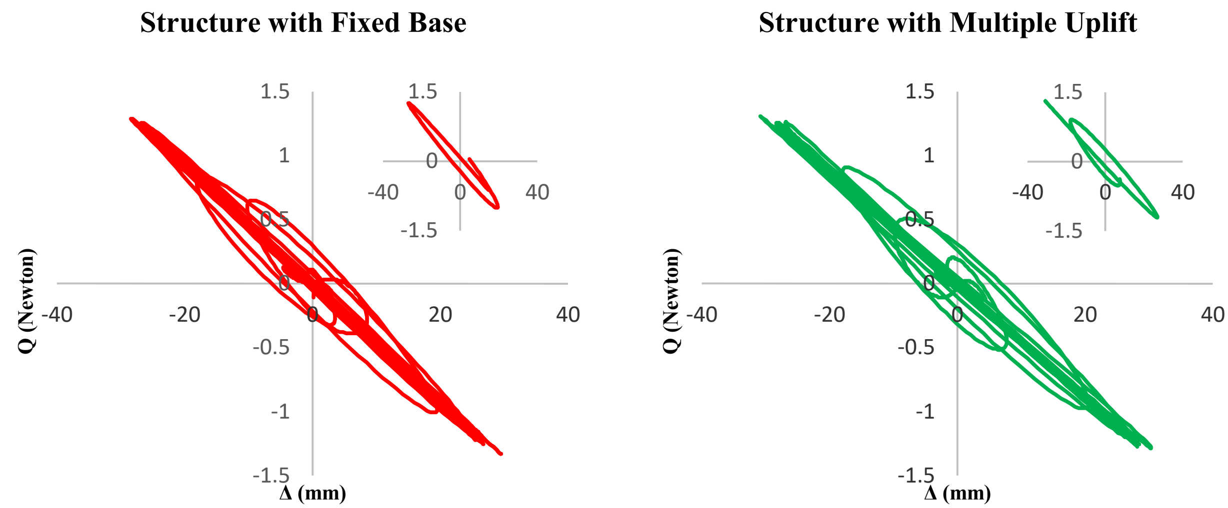


Figure 5: Experimental base shear-displacement (Q-Δ) curves along with an enlarged typical single loop

4. Conclusion

In order to study the dynamic behavior of the structure under harmonic loading particularly of effects of allowed columns uplift, a simple multi-degree of freedom single story prototype structure was investigated. The locally developed low-cost shake table was used to apply precise harmonic loading. Multiple uplifts were provided at a diagonal column of the structure. The following conclusions can be taken from this research:

1. In comparison to the applied loading on the base, the column response (averaged acceleration and displacement) is increased slightly at the column top.
2. Energy absorption (57%) is observed due to allowed multiple uplifts at column bases during applied harmonic loading.

The results obtained from this study show a significant effect of allowed column uplift in the column base connection. Numerical modeling on the mechanism of energy dissipation due to uplift is planned in parallel research. In addition, the multi-story steel frames structure with provided uplift at multiple stories should also be checked using a more complex shake table.

5. Acknowledgment

The authors would like to express our gratitude to every person/department who helped throughout the research work, particularly the EE department, Engr. Fayaz Khan, Engr. Nakhshab Ajaz, Mr. Irfan Waris and Mr. Osama Bin Rizwan. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

6. References

Ajaz, N., Rashid, A., & Ali, M. Relationship Between Energy Dissipation and Column Uplift in Simple 3D Structure During Dynamic Loading. *Structure*, *75*, 0-017.

Antonellis, G., Gavras, A. G., Panagiotou, M., Kutter, B. L., Guerrini, G., Sander, A. C., & Fox, P. J. (2015). Shake table test of large-scale bridge columns supported on rocking shallow foundations. *Journal of Geotechnical and Geoenvironmental Engineering*, *141*(5), 04015009.

Azuhata, T., Midorikawa, M., & Ishihara, T. (2008). Earthquake damage reduction of buildings by self-centering systems using rocking mechanism. In *The 14th World Conference on Earthquake Engineering*.

Baran, T., Tanrikulu, A. K., Dundar, C., & Tanrikulu, A. H. (2011). CONSTRUCTION AND PERFORMANCE TEST OF A LOW-COST SHAKE TABLE. *Experimental Techniques*, *35*(4), 8-16.

Benavent-Climent, A., Donaire-Avila, J., & Oliver-Sáiz, E. (2018). Seismic performance and damage evaluation of a waffle-flat plate structure with hysteretic dampers through shake-table tests. *Earthquake Engineering & Structural Dynamics*, *47*(5), 1250-1269.

Bird, J. F., & Bommer, J. J. (2004). Earthquake losses due to ground failure. *Engineering geology*, *75*(2), 147-179.

Ceccotti, A., Sandhaas, C., Okabe, M., Yasumura, M., Minowa, C., & Kawai, N. (2013). SOFIE project–3D shaking table test on a seven-storey full-scale cross-laminated timber building. *Earthquake Engineering & Structural Dynamics*, *42*(13), 2003-2021.

Çelebi, M., Bazzurro, P., Chiaraluce, L., Clemente, P., Decanini, L., DeSortis, A., ... & Milana, G. (2010). Recorded motions of the 6 April 2009 Mw 6.3 L'Aquila, Italy, earthquake and implications for building structural damage: overview. *Earthquake Spectra*, *26*(3), 651-684.

Clifton, C., Bruneau, M., MacRae, G., Leon, R., & Fussell, A. (2011). Steel structures damage from the Christchurch earthquake series of 2010 and 2011. *Bulletin of the New Zealand Society for Earthquake Engineering*, *44*(4), 297-318.

Cui, Y., Lu, X., & Jiang, C. (2017). Experimental investigation of tri-axial self-centering reinforced concrete frame structures through shaking table tests. *Engineering Structures*, *132*, 684-694.

Eduardo Reinoso, Miguel A. Jaimes & Luis Esteva (2017): Estimation of life vulnerability inside buildings during earthquakes, Structure and Infrastructure Engineering.

Elvin, A., & Uzoegbo, H. C. (2011). Response of a full-scale dry-stack masonry structure subject to experimentally applied earthquake loading. *Journal of the South African Institution of Civil Engineering*, *53*(1), 22-32.

Goggins, J., Broderick, B. M., Elghazouli, A. Y., Salawdeh, S., Hunt, A., Mongabure, P., & English, J. (2018, February). Shake Table Testing of Concentrically Braced Steel Structures with Realistic Connection Details Subjected to Earthquakes. In *Structures* (Vol. 13, pp. 102-118). Elsevier.

Huckelbridge, A. A., & Clough, R. W. (1977). *Earthquake simulation tests of a nine-story steel frame with columns allowed to uplift* (Doctoral dissertation, University of California, Berkeley).

Kulkarni, A. P., Sawant, M. K., & Shindepatil, M. S. (2017). Experimental Study using Earthquake Shake Table.

Lignos, Dimitrios G and Hikino, Tsuyoshi and Matsuoka, Yuichi and Nakashima, Masayoshi (2013) "Collapse assessment of steel moment frames based on E-Defense full-scale shake table collapse tests",Journal of Structural Engineering, Vol. 139.

Ma, X., Eatherton, M., Hajjar, J., Krawinkler, H., & Deierlein, G. (2010). Seismic design and behavior of steel frames with controlled rocking—Part II: Large scale shake table testing and system collapse analysis. In *Structures Congress 2010* (pp. 1534-1543).

Maikol Del Carpio, R., Mosqueda, G., & Lignos, D. G. (2019). Experimental investigation of steel building gravity framing systems under strong earthquake shaking. *Soil Dynamics and Earthquake Engineering*, *116*, 230-241.

Maqsood, Tariq & Schwarz, J. (2008). "Analysis of Building Damage during the 8 October 2005 Earthquake in Pakistan." *Seismological Research Letters*. 79. 163-177. 10.1785/gssrl.79.2.163.

Midorikawa, M., Azuhata, T., Ishihara, T., & Wada, A. (2006). Shaking table tests on seismic response of steel braced frames with column uplift. *Earthquake engineering & structural dynamics*, *35*(14), 1767-1785.

Nader, M. N., & Astaneh, A. (1991). Dynamic behavior of flexible, semirigid and rigid steel frames. *Journal of Constructional Steel Research*, *18*(3), 179-192.

Perrone, D., Calvi, P. M., Nascimbene, R., Fischer, E. C., & Magliulo, G. (2019). Seismic performance of non-structural elements during the 2016 Central Italy earthquake. *Bulletin of Earthquake Engineering*, *17*(10), 5655-5677.

Qin, X., & Chouw, N. (2010). "Experimental investigation of uplift effect on structures in earthquakes." *New Zealand society for earthquake engineering conference, Paper* (Vol. 14).

Rinaldin, G., & Fragiacomo, M. (2016). Non-linear simulation of shaking-table tests on 3-and 7-storey X-Lam timber buildings. *Engineering Structures*, *113*, 133-148.

Tafsirojjaman, T., Fawzia, S., Thambiratnam, D., & Zhao, X. L. (2019). Seismic strengthening of rigid steel frame with CFRP. *Archives of Civil and Mechanical Engineering*, *19*(2), 334-347.

Takagi, J., & Wada, A. (2019). Recent earthquakes and the need for a new philosophy for earthquake-resistant design. *Soil Dynamics and Earthquake Engineering*, *119*, 499-507.

Tohoku-Oki in 2011Inel, M., Cayci, B. T., & Meral, E. (2018). Nonlinear Static and Dynamic Analyses of RC Buildings. *International Journal of Civil Engineering*, 1-19.

Tomassetti, U., Correia, A. A., Candeias, P. X., Graziotti, F., & Costa, A. C. (2019). Two-way bending out-of-plane collapse of a full-scale URM building tested on a shake table. *Bulletin of Earthquake Engineering*, *17*(4), 2165-2198.

Wang, J. F., & Li, G. Q. (2007). Testing of semi-rigid steel–concrete composite frames subjected to vertical loads. *Engineering Structures*, *29*(8), 1903-1916.

Wiebe, L., Christopoulos, C., Tremblay, R., & Leclerc, M. (2013). Mechanisms to limit higher mode effects in a controlled rocking steel frame. 1: Concept, modelling, and low-amplitude shake table testing. *Earthquake Engineering & Structural Dynamics*, *42*(7), 1053-1068.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Performance Comparison of Circular and Rectangular Cross-Sectioned FRP Stirrups

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Abstract

Fiber-reinforced polymer (FRP) bars are being used as reinforcement in offshore concrete structures due to excellent corrosion resistant property. However, the pultruded FRP bar stirrups have limited performance due to reduced strength at bend parts and premature bond-slip failure at overlapping ends. Consequently, the strength and the ductility of concrete structural members are significantly reduced. The replacement of pultruded FRP bar stirrups of circular cross-section with continuously wound rectangular cross-sectioned stirrups could overcome the bond-slip failure and can significantly improve the bend strength (tensile strength of bed part). This paper presents an experimental study carried out to determine the efficiency of rectangular cross-sectioned FRP stirrups. Tests were conducted on six bend strength samples, two beam ($200 \times 500 \times 2650$ mm), and two column ($300 \times 300 \times 900$ mm) specimens to determine the bend strength, shear capacity, and confining performance of stirrups, respectively. Compressive strength of concrete used to cast the test specimen was 41.1 MPa. Bend strength ratio (ratio of the tensile strength of bend part to the straight part) of CFRP stirrups was improved from 0.34 to 0.52 by changing the circular cross-section to rectangular. A significant increase in shear strength of beam and ductility of the column was recorded for specimens with rectangular cross-sectioned stirrups.

Keywords
Stirrups, Column, Beam, FRP, Wet lay-up

1. Introduction

Fiber-reinforced polymer bars (FRP bars) are replacing conventional steel reinforcement in offshore concrete structures owing to the excellent resistance to corrosion and high strength to weight ratio. Pultruded FRP bars and stirrups are being used as flexural and shear reinforcement in bridge decks, beams, and retaining walls etc. Recently the researchers have also proved the viability of using FRP bars and stirrups in compression members (De Luca et al. 2009, Afifi et al. 2013, Tobbi et al. 2014, Karim et al. 2016). Despite the comparable performance of FRP stirrups with steel stirrups, pultruded FRP bar stirrups with circular cross-section have some shortcomings which affect their performance and make the structures uneconomical.

Pultruded FRP bar stirrups are fabricated by bending the FRP bars in required shape while the resin is yet not polymerized. The bending of bars causes the kinking of fibers at bend on the concave side of the bend (as shown in Figure 1). When these stirrups are stressed, the fibers on the convex side of bends are stressed more compared to inner ones, resulting in premature failure at bends (Lee et al. 2013, Dong et al. 2018). The ratio of tensile strength of stirrup at bend location to the tensile strength of straight part (bend strength ratio) of such stirrups ranges between 0.3 and 0.5 depending on the diameter of stirrup and bend radius (Ahmed et al. 2009, ACI 2015). Moreover, pultruded FRP bar stirrups are characterized by the overlapping at ends (see Figure 1). These stirrups undergo premature bond-slip failure after the concrete cover spalling. Consequently, the strength of stirrup's material is not fully utilized. Due to the above mentioned two shortcomings of pultruded FRP bar stirrups, higher amount of material is required for the desired design strength of structural members.

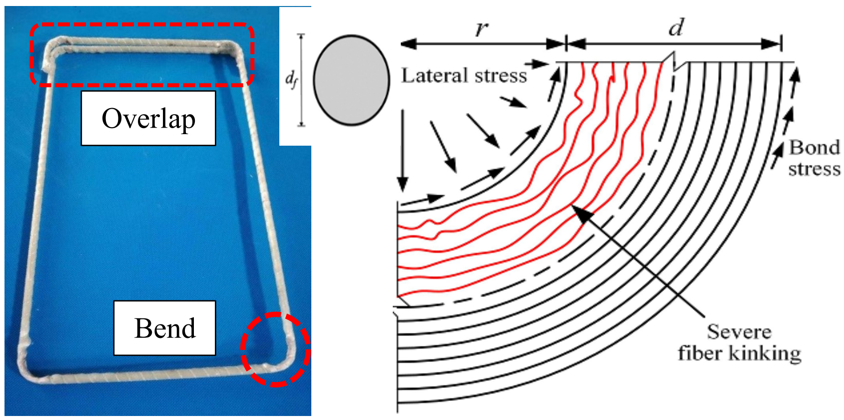


Figure 1: Kinking of fibers of pultruded bar stirrups at bends

The performance of FRP stirrups could be improved by reducing the kinking of fibers at bends and by overcoming the bond-slip failure at overlapping sections. Kinking of fibers can be reduced by changing the circular cross-section to rectangular (Lee et al. 2013). And the use of continuous winding stirrups can overcome the likelihood of bond-slip failure (Spadea et al. 2017, Dong et al. 2018). Figure 2 shows the rectangular cross-section stirrups with slight fiber kinking and without the requirement of overlap.

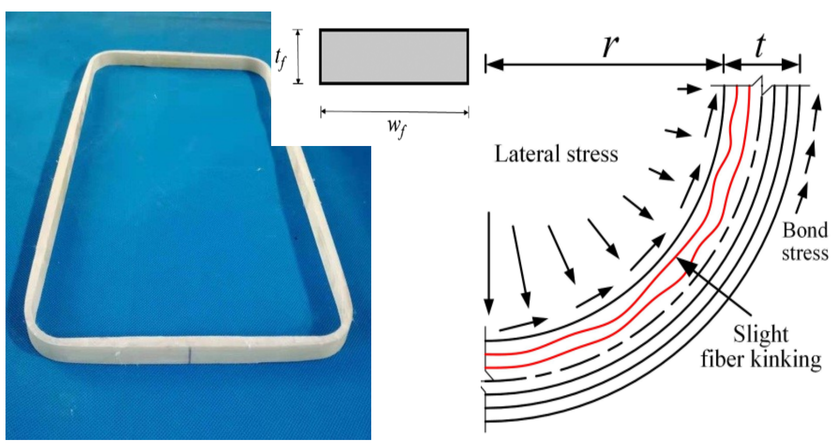


Fig. 2: Rectangular cross-section stirrups

In this experimental study, CFRP strip stirrups (CS stirrups), having rectangular cross-section are proposed to address the limitations of conventional CFRP bar stirrups (CP stirrups). Tests were conducted on bend strength samples, beam, and column specimens to determine the bend strength, shear capacity, and confining performance, respectively.

2. Fabrication of CS Stirrups

Rectangular cross-section stirrups can be fabricated in factory by winding the impregnated FRP fibers around the mandrel to make the tube and then cutting the tube to make stirrups of required width. This prefabrication technique requires mandrel according to the size and shape of stirrups and special arrangement for cutting the tube to make stirrups. However, for onsite fabrication of stirrups, wet-layup technique can be used to fabricate the rectangular cross-section stirrups. In this study, rectangular cross-section FRP stirrups were fabricated from CFRP strips cut from the unidirectional CFRP fabric. CFRP strips were impregnated with the epoxy resin and directly wound around the longitudinal bars of specimens. Figure 3 represents the diagram of the fabrication process of direct winding CS stirrups.

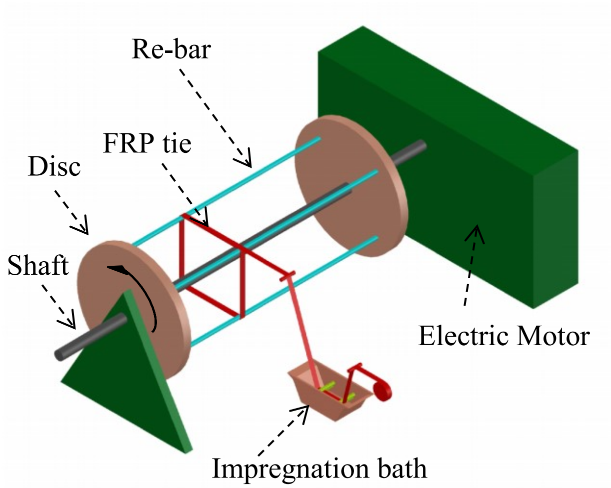


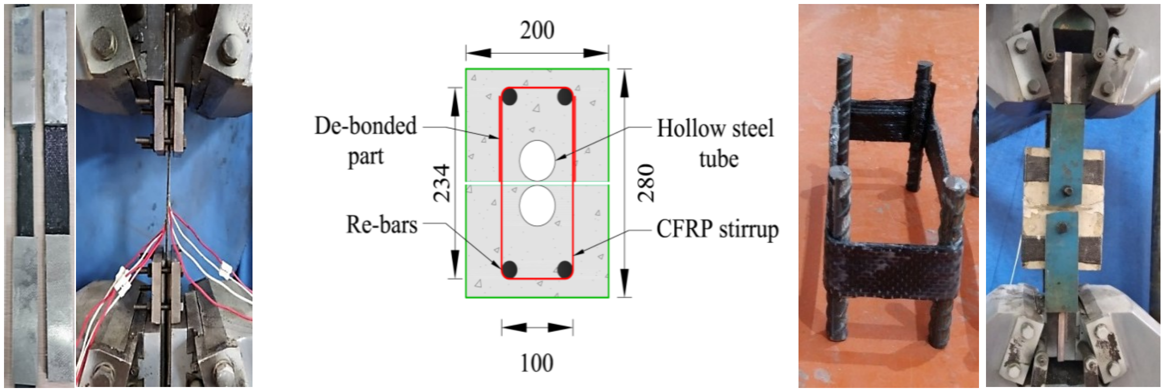
Figure 3: Representation of the fabrication process of FRP stirrups

3. Experimental Program

Three type of tests were conducted to study the performance of proposed CS stirrups; i) bend strength test, ii) shear strength test on beam specimens, iii) axial compression test on column specimens.

3.1 Bend-Strength Test

Tensile strength test and bend strength test were performed to compare the bend strength ratio of pultruded CFRP bar stirrups and CFRP sheet strip stirrups. Tensile strength of straight portion of stirrups was determined based on the specifications given by ISO 527-1(ISO 2012). Bend strength test was performed according to the provisions proposed in the study by Spadea et al. 2017. Figure 4 represents the details of tensile strength and bend strength tests specimens.

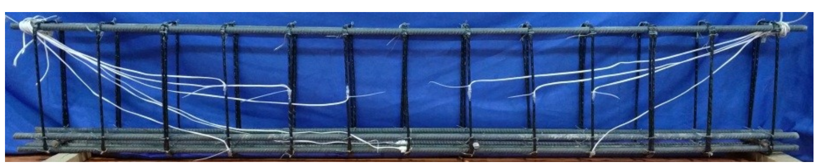


(a) (b)

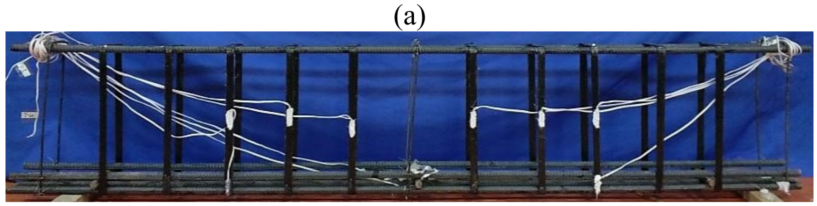
Figure 4: CFRP strip stirrups strength test (a) Tensile strength test, (b) bend strength test

3.2 Beam specimens

Two beam specimens (200 × 500 × 2600mm) were cast and tested to study and compare the shear performance of both type of CFRP stirrups (CP and CS stirrups). One specimen was reinforced with pultruded CFRP bar stirrups at spacing of 200 mm and the second specimen was reinforced with proposed CS stirrups with equal amount of shear reinforcement. Both the beam specimens were longitudinally reinforced with steel bars. Beam specimens were over designed in flexure to have desired shear failure. Figure 5 represents the reinforcement cages of beam specimens. Clear span of the beam was 2000 mm and a point load was applied at the mid span of the beam. Concrete having cylinder strength of 41 MPa was used to cast the column specimen.



(a)

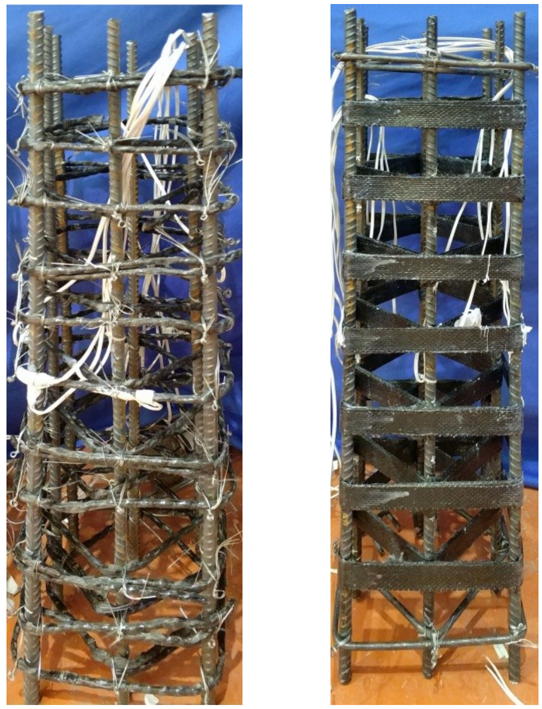


(b)

Figure 5: Reinforcement cages of beam specimens; (a) B-CP8, (b) B-CS20

3.3 Column specimens

Two column specimens (300 × 300 × 900 mm) were cast and tested to compare the confining efficiency of CFRP stirrups. One column specimen was reinforced with CP stirrups while other was reinforced with CS stirrups. Steel bars were used to longitudinally reinforce the column specimens. Figure 6 represents the reinforcement cages of column specimens. Concrete having cylinder strength of 41 MPa was used to cast the column specimen. Monotonic axial compression load was applied to test the columns.



(a) (b)

Figure 6: Reinforcement cages of columns (a) C-CP8 (b) C-CS30

4. Results and Discussions

Table 1 represents the results of bend strength specimens while Table 2 represents the results of beam and column specimens. From Table 1 it is clear that the bend strength ratio of CS stirrups is about 61% higher that than of CP stirrups. This improvement in bend strength ratio is due to the reduced kinking of fibers at bend portions for rectangular stirrups.

From Table 2 it can be seen that for beam specimens with same volumetric ratio of shear reinforcement, specimen with rectangular cross-section stirrups developed higher shear capacity compared to that of pultruded bar stirrups reinforced specimen. Similarly, for column specimens with CS stirrups, significant improvement in ductility (ultimate axial strain) was recorded compared to the specimen with CP stirrups. This improvement in ductility was due to the higher bend strength with zero possibility of bond-slip failure of CS stirrups.

Table 15: Bend strength test results

Material	Diameter or Width (mm)	Area (mm^2)	Elastic Modulus (GPa)	Tensile Strength (MPa)	Tensile Strain (%)	Bend Strength (MPa)	$(f_{fb}/f_{fu})_{exp}$ -
CFRP bar	8	50.2	150	f_{fu} = 1500	ε_f = 1.3	f_{fb} = 540	0.36
CFRP Strip	30	66.8	149	f_{fu} = 2200	ε_f =1.43	f_{fb} =1067	0.48
CFRP Strip	20	50.1	146	f_{fu} = 1908	εf =1.26	f_{fb} =1115	0.58

Table 16: Beam and column test results

Beam specimen		
Specimen ID	B-CP8	B-CS20
Stirrups type	CFRP bar stirrups	CFRP strip stirrups
Cross-section dimensions of stirrups (mm)	Φ8	20×2.5
Spacing of stirrups (mm)	200	200
Maximum shear capacity, V_n (kN)	368.5	392.5
Maximum deflection corresponding to V_n (mm)	11.27	13.81
Column specimen		
Specimen ID	C-CP8	C-CS30
Stirrups type	CFRP bar stirrups	CFRP strip stirrups
Cross-section dimensions of stirrups (mm)	Φ8	30×2.22
Spacing of stirrups (mm)	75	100
Volumetric ratio of stirrups (ρ_v)	1.87	1.87
Confinement efficiency (fcc'/fco')	1.34	1.31
Ultimate axial strain (%)	1.11	2.82

5. Conclusions

This paper presents the results of an experimental study carried out to examine the performance of proposed CFRP strip stirrups. Tests on beam, column, and bend strength specimens were performed to evaluate the shear and confining performance of CS stirrups. Based on the observations and results, following conclusion can be drawn:

- Bend strength ratio of CFRP Strip stirrups was 61% higher than that of CP stirrups.
- Shear capacity and ductility of beam and column specimens were improved by 6.5% and 154%, respectively, for CS stirrups reinforced specimens.

5. Acknowledgement

This work was supported by the National Key Research and Development Program of China (No. 2017YFC0703001) and the National Natural Science Foundation of China (No. 51408153, No. 51478143, and No. 51278150).

6. Reference

ACI (2015). ACI 440· 1R-15: Guide for the design and construction of concrete reinforced with FRP bars, ACI Farmington Hills, Michigan.

Afifi, M. Z., H. M. Mohamed and B. Benmokrane (2013). "Axial capacity of circular concrete columns reinforced with GFRP bars and spirals." *Journal of Composites for Construction* 18(1): 04013017.

Ahmed, E. A., A. K. El-Sayed, E. El-Salakawy and B. Benmokrane (2009). "Bend strength of FRP stirrups: Comparison and evaluation of testing methods." *Journal of composites for construction* 14(1): 3-10.

De Luca, A., F. Matta and A. Nanni (2009). "Behavior of full-scale concrete columns internally reinforced with glass FRP bars under pure axial load." *Composites & Polycon*: 15-17.

Dong, H.-L., D. Wang, Z. Wang and Y. Sun (2018). "Axial compressive behavior of square concrete columns reinforced with innovative closed-type winding GFRP stirrups." *Composite Structures* 192: 115-125.

ISO, E. (2012). "527-1: 2012." Plastics–Determination of tensile properties–Part 1.

Karim, H., M. N. Sheikh and M. N. Hadi (2016). "Axial load-axial deformation behaviour of circular concrete columns reinforced with GFRP bars and helices." *Construction and Building Materi*als 112: 1147-1157.

Lee, C., M. Ko and Y. Lee (2013). "Bend strength of complete closed-type carbon fiber-reinforced polymer stirrups with rectangular section." *Journal of Composites for Construction* 18(1): 04013022.

Spadea, S., J. Orr and K. Ivanova (2017). "Bend-strength of novel filament wound shear reinforcement." *Composite Structures* 176: 244-253.

Spadea, S., J. Orr, A. Nanni and Y. Yang (2017). "Wound FRP shear reinforcement for concrete structures." *Journal of Composites for Construction* 21(5): 04017026.

Tobbi, H., A. S. Farghaly and B. Benmokrane (2014). "Behavior of Concentrically Loaded Fiber-Reinforced Polymer Reinforced Concrete Columns with Varying Reinforcement Types and Ratios." *ACI Structural Journal* 111(2).

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

OUT-OF-PLANE BEHAVIOR OF MORTAR-FREE INTERLOCKING SOLID-WALLS UNDER SEISMIC LOADING – A REVIEW

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Abstract
Mortar-free interlocking construction have gained popularity due to its speedy erection and energy dissipation. However, its progress at local level in developing countries is quite limited from dynamics point of view. The in-plane and out-of-plane behaviors of solid-walls play an important role to avoid overall structure collapse by resisting earthquake forces. Out-of-plane behavior is crucial than the in-plane behavior. In this paper, an effort is being made to understand the out-of-plane behavior of mortar-free interlocking solid-walls under seismic loading through literature research. Both full scale and scaled-down walls are considered. Different types of interlocking solid-walls studied by different researchers through physical testing are being discussed along with their testing mechanism and studied parameters. Special focus is made on exploring the behavior considering their boundary conditions, loading types, wall thickness to height ratio, and use of hollow or solid interlocking blocks etc. The outcome of this work is the selection of a feasible mortar-free interlocking solid-wall for consideration in local construction.

Keywords
Mortar-free interlocking, Out-of-plane behavior, Full scale and scaled-down walls, Seismic loading

1. INTRODUCTION

Earthquake is a natural disaster which produces strong ground motion Major effects of earthquake cause severe damages, such as failure of building, roads and bridges, which may affect many people. Structure is frequently affected during forceful earthquake and flop. Most of Structures are often affected during intense earthquake and collapse. Earthquake badly effect masonry structures due to strong ground motion. Earthquake in Pakistan occupied Kashmir region in 2005, caused more than 72000 causalities, more than 68000 injuries, more than 450000 buildings damages and the losses to a total cost of US $5.2 billion. (Rossetto and Peiris, 2009). During the earthquake of L'aquila, the city of Central-Italy in 2009 damages about 10000 buildings, 328 deaths and more than 1500 injuries (Brandonisio *et al*., 2013). Earthquake in Nepal in 2015, 1, 50000 peoples dislocated after the severe damages of the earthquake (Chen *et al*., 2016). The major reason of the damage of masonry buildings is the use of typical un-constrained masonry system. In-addition, during the earthquakes the majority of brick masonry buildings are collapsed due to the insufficiencies in design and there execution. The earthquake in Doğubayazıt, the district of Province Ağrı of Turkey, caused 1000 masonry building damages and 100 houses badly affected (Bayraktar *et al*., 2007). In the earthquake of Christchurch in 2011, about 650 unreinforced buildings of clay brick masonry and 90 unreinforced buildings of stone masonry were damaged (Dizhur *et al*., 2011). More than 25000 peoples were homeless due to the severe damages of unreinforced masonry buildings in the earthquake of Abruzzo in 2009 (Indirli *et al*., 2013). Unreinforced-masonry buildings are being classified as the most of the earthquake prone constructions in New Zealand (Russell and Ingham, 2010). In the earthquake of

Haiti in 2010, most of the unreinforced masonry structures were damaged severely (Desroches *et al*., 2011).
Earthquake-resistant and cost-effective houses are the need of this era for the earthquake-prone regions in developing countries. Due to the lack of earthquake-resistant development techniques, the developing countries are suffering of severe structure damage and social loss during the earthquakes. However, literature shows that different earthquake-resistant development methods and techniques have been adopted for the said purpose. For example, provide the plinth & lintel beams, vertical stiffeners in masonry structures. Ali *et al*. (2013) proposed mortar-free interlocking-block structure as modern construction technique for the earthquake-resistant housing. Mortar-free block structure of coconut-fiber reinforced interlocking with post-tensioned ropes of coconut fiber was being tested for the dynamic loading (Ali, 2017). Out-of-plane behavior of unreinforced-masonry structures is more crucial than the in-plane behavior. Most of the damages of masonry structures have been occurred due to the out-of-plane failure. Sometimes, no or poor anchorage of walls with diaphragm causes the severe damages. Out-of-plane failure is more critical and it appeared very quickly. Out of 182 damages to unreinforced masonry cavity wall construction, 72% damages were due to the out-of-plane failure 28% were caused by the in-plane failure (Giaretton *at al*., 2016). During the earthquake of Darfield, Christchurch in 2010, the out-of-plane failure of walls was the first one to appear on the television-screen right after the earthquake (Ingham and Griffith, 2015). As shown in figure 1, the primary reasons of such brick-masonry collapses were stated as lowly construction, poor materials usage, non-designed building walls, gable walls without confinement, and cracking started from edges of the openings. All the damages were occurred due to the out-of-plane failure of the said structures. Some examples of out-of-plane failures in solid masonry walls are shown below in figure 1. Different types of wall collapse and corner failure are shown.

(a)

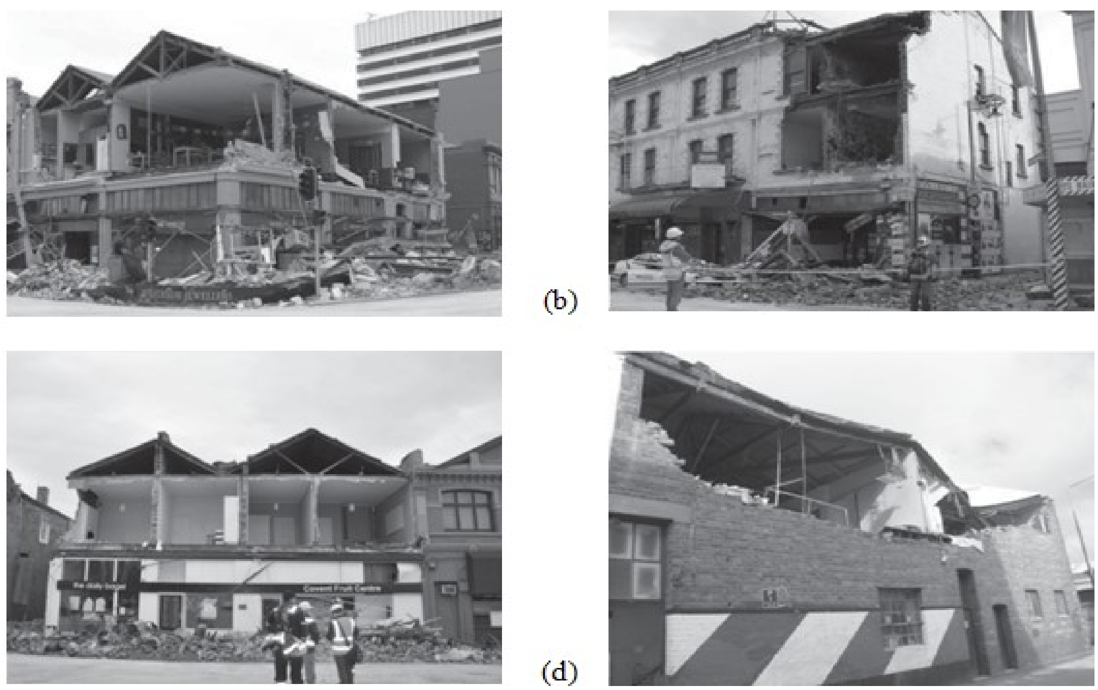

Figure 1: Masonry wall damages (Ingham and Griffith, 2015)**; (a) corner failure; (b) wall collapse in 2nd and 3rd story; (c) wall collapse at 1st story; and (d) out-of-plane collapse**

2. DAMAGES OF TYPICAL MASONRY-STRUCTURES DURING GROUND-MOTION

Damages to typical unreinforced masonry structures during the earthquakes are being stated by different researchers from all over the world. Most of the damages to masonry structures occurred due to their un-confinement and poor anchorage with diaphragm. Sharma *et al*. (2016) lead survey study after the Gorkha earthquake in Nepal in 2015. Approximately 80000 partially or fully damage buildings were reported.

Jagadish *et al*. (2003) carried out a study for the behavior of unreinforced-masonry structures during the Bhuj earthquake in India in 2001. It is concluded in the study that most of the masonry building of mud mortar or lime mortar were being severely damages due to the low bond strength. According to the study, masonry buildings with cement mortar resist more than the others due to the strong bonding. Use of lintel band and provision of steel reinforcement in corners and junctions of masonry structures were being suggested as the future recommendation in the study. The research recommended that, although the provision of lintel bands will reduce the in-plane failure of masonry walls but it will not be helpful during the occurrence of out-of-plane flexural failure. Particularly, in the flexural cracks which propagates horizontally and ultimately results the out-of-plane failure. Lintel-band failure and corner failure due to the out-of-plane failure are shown below in Figure 2a and Figure 2b respectively.

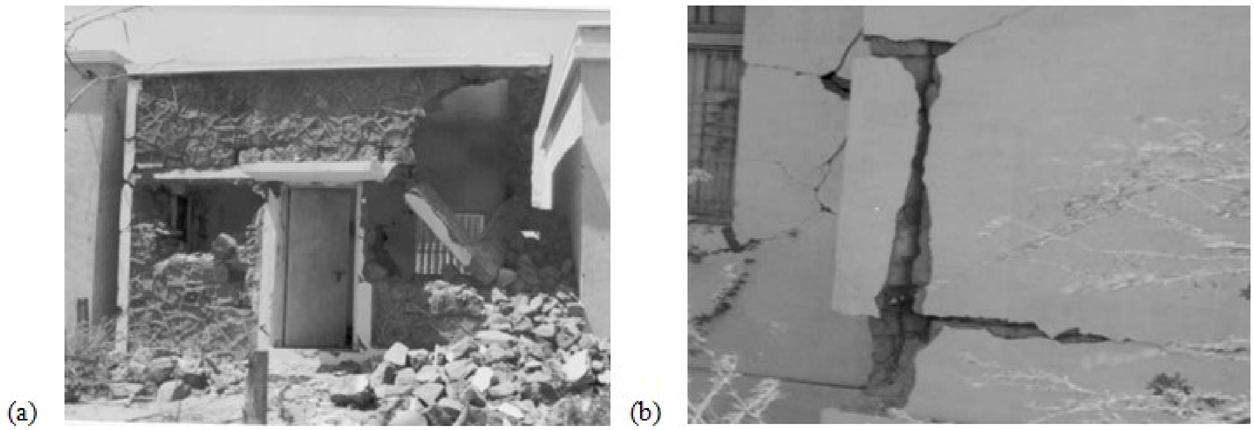

Figure 2: Damages in walls having horizontal and vertical stiffeners (Jagadish *et al*., 2003)**; (a) lintel-band failure; and (b) corner reinforcement failure**

Javed *et al*. (2006) conducted a research on the behavior of masonry structure after the earthquake of 2005 in Pakistan occupied Kashmir. The study included that the majority of damages were occurred due to the shear forces produces during the in-plane behavior in walls and out-of-plane flexure of the walls. Most of the in-plane diagonal cracks and the X-diagonal cracks were the results of shear forces produces in the plane of the wall. Typical diagonal cracks originating from the corners of the opening due to non-provision of the corner reinforcement. Toe crushing failure of masonry piers occurred due to the cyclic nature of seismic forces. Inertial forces were the main cause of collapse of masonry houses due to the out-of-plane failure. Out-of-plane overturning of gable walls was also noted in different localities. Diagonal compressive forces caused the non-structural masonry walls. Collapse of masonry bridges and water tank on the roof of houses was also reported. Poor quality of cement mortar and the undressed stone masonry was concluded as the major reason behind the all said damages and collapses.

3. NEW APPROACH FOR EARTHQUAKE-RESISTANT STRUCTURES

Ali (2017) studied the impact of post-tensioned ropes of coconut-fiber in governing uplift of interlocking mortar-free blocks-construction during earthquake loading. It was stated that suggested interlocking block shown in Fig. 3 is accomplished of regaining its first position after the ground motion due to providing inclined key shape in the block. Investigational results were used to improve the empirical relation in the form of function of peak ground acceleration. A difference of 35% was witnessed in predicting the actual seismic response of the structure, which may obey due to the difficulty of the interlocking block column. Results of the study appeared satisfactory in-order to have cost-effective earthquake-resistant houses construction techniques. A coconut-fiber-reinforced-concrete interlocking block is shown in Figure 3, proposed by (Ali, 2017).

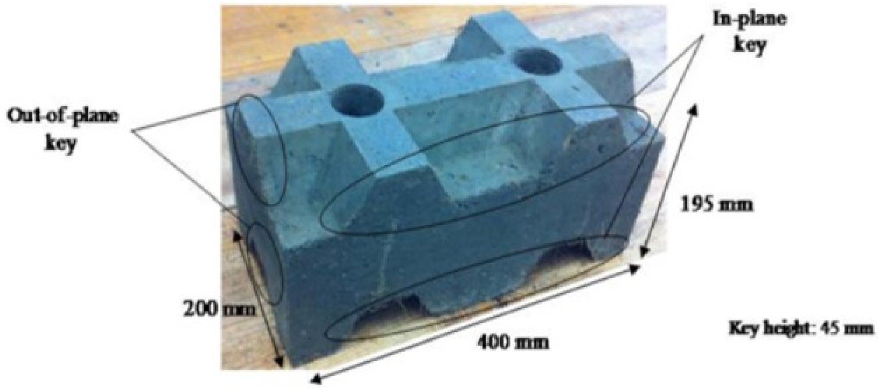

Figure 3: Coconut-fiber-reinforced-concrete interlocking block (Ali 2017)

Qamar *et al*. (2020) carried out a study for the improvement of lateral resistance in mortar-free interlocking walls with plaster by using nature fibers. The major reason of the failure of mortar-free interlocking wall system is the out-of-plane lateral resistance. Increase in lateral peak load is noted in this study and further increase also noted for rice straw and sisal fiber reinforced plastered wall system. Khan (2019) suggested use of interlocking plastic-blocks for earthquake proof houses due to their less-weight with the combination of energy dissipation because of uplift of blocks. Liu *et al*. (2016) studied the cyclic behavior of non-interlocking mortar-less brick and interlocking mortar-less brick. During the study of cyclic behavior, the effects of different interlocking forms, loading compression stress levels and loading cycles were considered. With the help of hysteresis loop method, a mechanical model was established. The shear failure modes of all of the tested joints were well-defined by using mohr-coulomb failure method. Increase in the loading cycle, decrease in the friction coefficients of all of the joints was observed carefully. With the reduction of the flatness of the surface of the interlocking, a significant increament has been seen in the degradation of the friction.

4. OUT-OF-PLANE BEHAVIOR OF MORTAR-FREE INTERLOCKING SOLID-WALLS

Out-of-plane behavior is more critical than the in-plane behavior. Many researchers reported that the most of the damages and collpase in unconfined masonry structures had been occurred due to the failure in out-of-plane bahavior. Various studies has been conducted on the out-of-plane behavior effects. Kallioras *et al*. (2018) delivered a single data set that captures at full-scale the in-plane and out-of-plane performance of un-reinforced masonry wallig. And provided a dynamic global response of a building under earthquake loading. Saifee *et al*. (2011) concluded that the wall's behavior was majorily controlled by large horizontal displacement and dry-joint opening about at mid hight of the wall (location of extreme moment). Figure 4 shows the interlocking mortor less wall exposed to out-of-plane loading. Martinez and Atamturktur (2018) mortar-less masony wall was experienced under out of plane loading to understand the performance of wall. Enhancement in crucial lateral load capacity was assessed because of rise in block compressive-strength and also effect of wholly or partly grouted wall was compared. Out-of-plane response of mortar-free walls by different researchers are shown in Table 1. Diverse studies had been carried out to examined the ot-of-plane behavior of mortar-free walls by the different researchers.

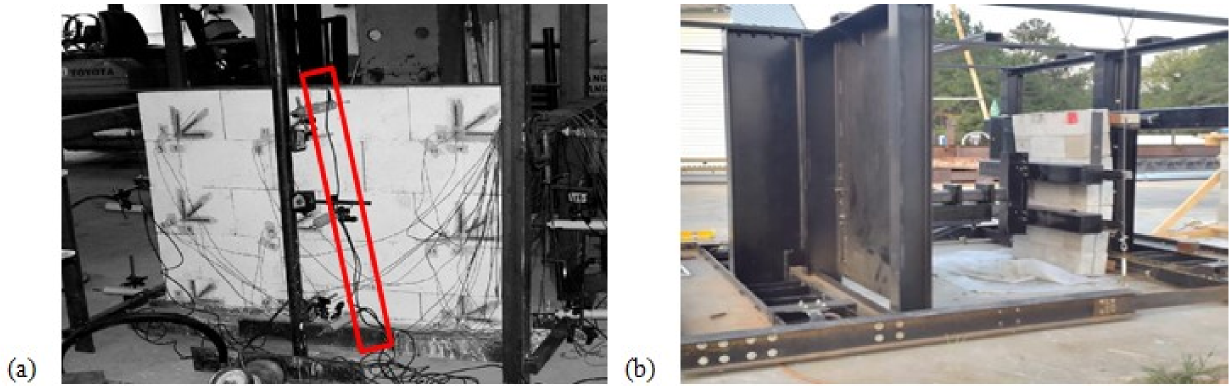

Figure 4: Out-of-plane testing of mortar-free wall; (a) Saifee *et al*. (2011) and **(b)** Martinez and Atamturktur (2018)

Table 1: Out-of-plane response of mortar-free walls by different researchers

Reference	Study	Conclusions
Ali et al. (2013)	Dynamic behavior of mortar-free interlocking structures	Mortar free walls were collapsed in bending in out-of-plane direction as one of the bottom interlocking keys broken and partly shear-off, producing uncertainty in the structures
Saifee et al. (2011)	Experimental study was examined to investigate the structural behavior of mortar less interlocking load bearing hollow block wall under out of plane loading	Out-of-plane load capacity, mode of deformations and dry joint opening in the wall were extremely affected by grout and the reinforcement and pre compressive load
Thanoon et al. (2007)	Structural response of mortar less interlocking masonry system under the eccentric compressive loads	Interlocked walling system is a possible alternate to conventional brick-mortar-masonry as it showed better and comparable structural presentation under the axial and eccentric loading

5. CONCLUSIONS

Typical unreinforced masonry structures are prone to earthquake loading and seismic damages. Developed countries has adopted the confined masonry structures technique to avoid. However, confined masonry structures are also not enogh to avoid the severe damages and collpase of building and structures during the earthquake. In developing countries, it is the need of time to adopt the modern techniques for the earthquake resistant housing. For this purpose, the researchers from all over the world are focusing on the new technique of mortar-free interlocinkg structures. The provided literature has the significant studies of the researchers which proposed the different typpes and sizes of interlocking mortar-free structures. Behavior of these interlocking mortar-free block prototypes against earthquake loading can be predicted well by considering the small scale testing. Their analytical endorsement can be used to improve empirical relations in order to perform basic testing with the identification of error percentages.

A lot of researches support and authenticate the results get from the testing of these prototype structures. In present, most of the researchers have motivated on concrete block or masonry block approache. But at the same time, the usage of any other light-weight material can play a dynamic role in decreasing the inertial forces.

6. RECOMMENDATIONS

Out-of-plane behavior of mortar-free interlocking plastic block solid-wall should be examined in laboratory by using the shake table. Empirical relations and equations for the better understanding of testing with boundary conditions and to bridge the gap of real condition and prototype testing needs to be explored in further detail in near future. Arrangement of test setups for the accuracy in test results plays an important roles inm the experimental program of a research. The aspect regarding to the accuracy of test setup is need to be explored. Commercialization and making of various interlocking blocks is difficult because of the non-engineered approach of confined contractors, absence of machinery and well-trained labor. The aspect regarding the commercialization needs to be investigated to make these blocks accessible commercially for the usage of construction engineering. Skilled labour for this pupose is also an essential part.

7. ACKNOWLEDGEMENTS

The authors would like to acknowledge CUST library facility for providing relevant literature which helped a lot in smooth conduct of this study. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

REFERENCES

Ali, M. (2017) “Role of Post-tensioned Coconut-fibre Ropes in Mortar-free Interlocking Concrete Construction During Seismic Loadings”. *KSCE Journal of Civil Engineering* (0000) 00(0):1-8.

Ali, M., Briet, R., and Chouw, N. (2013) “Dynamic response of mortar-free interlocking structures”.*Construction and building materials*, (42), 168-189.

Brandonisio, G., Lucibello, G., Mele, E., Lucs, A.D. (2013) “Damage and performance evaluation of masonry churches in the 2009 L’Aquila earthquake”. *Engineering Failure Analysis*, Volume 34, Pages 693-714.

Bayraktar, A., Coskun, N., and Yalcin, A. (2007) “Damages of masonry buildings during the July 2, 2004 Dog˘ubayazıt (Ag˘rı) earthquake in Turkey”. *Engineering Failure Analysis* 14 147–157.

Chen, H., Xie, Q., Li, Z., Xue, W., Liu, K. (2016) “Seismic damage to structures in the 2015 Nepal earthquake sequences”. ISSN: 1363-2469 (Print) 1559-808X.

Dizhur, D., Ingham, J., Moon, Lisa., Griffith, M., Schultz, A., Senaldi, I., Magnes, G., Dickie, J., Lissel, S., Centeno, J., Ventura, C., Leite, J., and Lourenco, P. (2011) “Performance of masonry buildings and churches in the 22 february 2011 christchurch earthquake”. *Bulletin of the New Zealand society for earthquake engineering*, vol. 44, no. 4.

Desroches, R., Comerio, M., Eberhard, M., Mooney, W., and Rix, G.J. (2011) “Overview of the 2010 Haiti Earthquake,” *Earthquake Spectra*, vol. 27, no. 1. pp. 124-130.

Giaretton, M., Dizhur, D., Porto, F. D., AND Ingham, J. M. (2016) “Construction details and observed earthquake performance of unreinforced clay brick masonry cavity-walls”. *Structures*, S2352-0124(16)30009.

Ingham, J., and Griffith. M. (2015) “Performance of Unreinforced Masonry Buildings During the 2010 Darfield (Christchurch, Nz) Earthquake”. *Australian Journal of Structural Engineering*. Vol 11 No 3.

Indirli, M., Alexandros, L., Formisano, A., Borg, R.P., and Mazzolani, F.M. (2013) “Seismic Damage Assessment of Unreinforced Masonry Structures After The Abruzzo 2009 Earthquake: The Case Study of the Historical Centers of L'Aquila and Castelvecchio Subequo". *International Journal of Architectural Heritage*, 7: 536–578.

Jagadish, K. S., Raghunath, S., and Nanjunda, K. S. (2003) "Behaviour of masonry structures during the Bhuj earthquake of January 2001," *Journal of Earth System Science*, vol. 112, no. 3, pp. 431-440.

Javed, M., Khan, A. N., Penna, A., and Magenes, G. (2006) "Behaviour of masonry structures during the kashmir 2005 Earthquake". *First European Conference on Earthquake Engineering and Seismology*, Paper Number:1077.

Khan, F. (2019) "Dynamic Behavior of Prototype Interlocking Plastic-block Structure Using Locally Developed Low-cost Shake Table," MS Thesis, *Department of Civil Engineering, Capital University of Science and Technology, Islamabad, Pakistan*.

Kallioras, S., Guerrini, G., Tomassetti, U., Peloso, S., and Graziotti, F. (2018) "Dataset from the dynamic shake-table test of a full-scale unreinforced clay-masonry building with flexible timber diaphragms". *Data in Brief* (18), 629-640.

Liu,H., Liu, P., LiN, K., and Zhao, S. (2016) "Cyclic Behavior of Mortarless Brick Joints with Different Interlocking Shapes". *Materials* 2016, 9, 166; doi:10.3390/ma9030166.

Martínez, M., Atamturktur,, S. (2018) "Experimental and numerical evaluation of reinforced drystacked Concrete masonry walls". *Journal of Building Engineering*, S2352-7102(17)30579-X.

Qamar, F., Thomas, T., and Ali, M. (2020) "Improvement in lateral resistance of mortar-free interlocking wall with plaster having natural fibres". *Construction and Building Materials* (234).

Russell, A.P., and Ingham, A.M. (2010) "prevalence of New Zealand's unreinforced Masonry buildings". *Bulletin of the New Zealand society for earthquake engineering*, vol. 43, no. 3.

Rossetto, T., and Peiris, N. (2009) "Observations of damage due to the Kashmir earthquake of October 8, 2005 and study of current seismic provisions for buildings in Pakistan". *Bull Earthquake Eng*, 7:681–699.

Sharma, K., Deng, L., and Noguez, C.C. (2016) "Field investigation on the performance of building structures during the April 25, 2015, Gorkha earthquake in Nepal". *Engineering Structures*, Volume 121, Pages 61-74.

Safiee, N.Z., Jaafar, M.S., Alwathaf, A.H., Noorzaei, J., and Abdulkadir, M. R. (2011) "Structural Behavior of Mortarless Interlocking Load Bearing Hollow Block Wall Panel under Out-of-Plane Loading". *Advances in Structural Engineering*, Vol. 14, No. 6.

Thanoon, W.A., Jaafar, M. S., Kadir, M. R. A., Ali, A. A. A., Trikha, D. N., and Najm, A. M. S. (2004) "Development of an innovative interlocking load bearing hollow block system in Malaysia". *Construction and Building Materials* 18 (2004) 445–454.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Analyzing the Mobility of Multilane Highway using Travel Time as a Performance Measure

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Abstract: Rapid urbanization and open accessibility along national highways have impacted their mobility to a great deal in Pakistan. In this study a multilane highway section on N-5 GT road is examined for its mobility function. Moreover, performance measures like travel time and speed that are not very common in practice in Pakistan were used for analysis as they cater both agency and user demand. Travel time and traffic count data is collected using video recordings at three prescribed locations that were later evaluated based on License plate matching technique. Speed and travel time parameters like time mean speed, space mean speed, travel rate, Speed reduction index and travel time reliability were also calculated to better understand the actual situation and behavior of traffic on selected highway section. It was observed that majority of the traffic is using the highway for intracity movement rather than intercity movement based on the vehicles observed at the start and at the end of section. Moreover, average speed of vehicles was considerably lower than the posted speed limit that also confirms that the overall mobility of highway is on lower side which is a function of collector or local roads.

Keywords: Mobility, Multi-lane highway, Travel time, license plate technique

1. Introduction and literature review

Mobility refers to the through portion of trips and is most affected by supply and demand, land use and other factors on the road. In highway systems, mobility is provided by high-type facilities, such as freeways, expressways, and other multilane divided highways. But due to rapid urbanization change in land use patterns and open accessibility along highways the mobility of multilane highways has been greatly reduced specifically in under developing countries like Pakistan.

According to highway capacity manual multilane highways are divided highways with a minimum of two lanes in each direction. Their speed limits usually range from 60-100 Kmph with zero or partial control of access (Manual, 2010). These roadway facilities usually provides through service that ranges from 60 to 100 percent depending upon either it is a freeway or major or a minor arterial (Roess, Prassas, & McShane, 2011).

Similarly a study conducted by (Fitzpatrick, 2003) suggested that multilane divided highway passing through a urban area having level or rolling terrain have anticipated operating speed , anticipated posted speed and design speed as 80-100 ,70-90 and 70-100 kmph respectively. This shows that multilane highways are usually designed as through facility to provide greater mobility and little accessibility.

Mobility can incorporate several quantitative elements such as riding comfort, absence of speed changes but most basic function is operating speed and travel time (Hancock & Wright, 2013). There are many mobility performance measures for highways that are currently being used all over the world. Travel Time is relatively new and modern method being used in more developed countries that caters for both user and travel agency requirements (Lasley, Lomax, Eisele, & Schrank, 2014).

A project carried out by National Cooperative Highway Research Program "Quantifying Congestion" recommended that mobility and congestion data should be estimated and presented based on travel time-based measures. Furthermore, measures such as travel speed, travel time, travel rate and travel delay can also be used as it stratifies the needs of road users. Another measure recommended by the study is speed reduction index (SRI) that reflects the ratio of the relative speed change between congested and free-flow traffic conditions. It has continuous scale with numerical values between 0 and 10.

$$SRI = \left(1 - \left[\frac{actual\ speed}{free\ flow\ speed}\right]\right) \times 10 \qquad (1)$$

Similarly Travel rate also reflects the link average congestion level that is the rate of motion, in min/km, for a roadway section. It is calculated by dividing the segment travel time by the segment length.

$$TR = \frac{section\ travel\ time}{section\ length} \qquad (2)$$

(Ter Huurne & Andersen, 2014) quantified congestion on major arterials in Stellenbosch, South Africa. Based on speed reduction and congestion indices study noticed that there are too many vehicles on the road network at specific hours of the day. The level of congestion on the arterials is worse in the morning as compared to the afternoon that was due to university and school traffic.

Travel time reliability is also of great concern for road travelers and transportation planners. Federal Highway Administration has specified measures that describe time reliability in slightly different ways including Travel Time Index, Buffer Index, Planning Time and Planning Time Index.

A research effort by (Mehran & Nakamura, 2007) applied travel time reliability as a measure of performance on two different segments of an intercity expressway in japan .Travel time indexes varies from time to time according to demand fluctuations and traffic influencing events but it never reaches one in any of the sections.

Likewise a study conducted by (Bharti, Sekhar, & Chandra, 2018) in India used travel time reliability measures such as planning time, buffer time , planning time index and buffer time index for evaluating urban arterial and intercity highway corridors. The study concluded that for travel time reliability of 95% average planning time was slightly lower on non-working days when compared to working days on interstate highway however substantial difference was observed during non-working days when compared to working days on urban arterial corridors.

Travel time can be used to calculate the Speed, but there are several types of speeds that should be distinguished. Two of the most important speed values are time-mean speed (TMS) (3) and space mean speed (SMS) (McShane & Roess, 1990)

$$TMS = \frac{\sum v_i}{n} \quad (3)$$

$$SMS = \frac{d}{\frac{\sum t_i}{n}} \quad (4)$$

Time-mean speed is related with a single point along a highway over time, whereas the space-mean speed is related with a predetermined length of roadway. SMS are usually used when reducing and investigating travel time data (Turner, Eisele, Benz, & Holdener, 1998)

Several data collection techniques can be used to measure travel times. However, measuring travel time using video cameras and manually transcribing license plates using human observers has a moderate to high accuracy and it is less costly as well. Moreover it provides travel times from a large sample of drivers, which is useful in identifying variability of travel times among vehicles within the traffic stream and at the same time has the capability to investigate short time periods as low as 15 mins averages for continuous data (Turner et al., 1998). A research study by (Berry, 1952) found that for a given roadway segment sample sizes from 25 to 102 license matches are necessary. Likewise Manual of Transportation Engineering Studies also reported a minimum sample size of 50 (Hummer, Robertson, & Nelson, 1994). Therefore, this study evaluates the mobility function of multilane national highway (N-5) that was designed primarily for through movement with open access to the surrounding areas. Performance measures like travel time and speed that are not very common in practice in Pakistan were used as they cater both agency and user demand. Speed and travel time parameters like space mean speed, speed reduction index, travel rate was also calculated to better understand the actual situation and behavior of traffic on the highway. Travel time reliability measures were also analyzed to access the stability in travel time of a trip on the highway .Whereas the travel time is collected based on license plate matching technique as it provides more variability in data and less costly as well.

2. Study Section

Study section was selected in northern region of national highway N-5 (Multilane divided highway) between two major cities of Islamabad and Taxila. As it is a national highway, so drivers expect to travel at relatively high speeds. Speed limit on the section varied between 50-100 for LTV whereas there was open accessibility along the highway section. The road section was so selected that it is free from the effect of intersections and in plain terrain with no significant horizontal and vertical curves. Total length of study section was 9 km having two lanes in travel direction, moreover study section has semi-urban roadside development characteristics.

3. Methodology and Field Data Collection

All field data was collected in pre-planned format by three teams of two members in each group under supervision of the author. The traffic data was collected on typical weekdays between 12:00pm - 2: 00pm and Video photographic survey method was adopted for collection of field data. Three observation points were established for installing the video cameras: start of section (Station 1), end of section (Station 3) and mid-block (Station 2) for recording the entry, exit and traffic count of vehicles respectively. Station 1 and 3 cameras were located immediately adjacent to the right shoulder in the median from where the license plate number can be easily readable whereas station 2 camera is at elevated point focusing camera on carriageway. Data was recorded in four sessions within two hours and the errors due to time lag at the time of extraction were avoided by synchronizing the recordings. Classified traffic count and travel time data is then manually transcribed into data sheets using human observers by playing the video files on computer. All the vehicles are classified into six categories namely Car, Van, Bus, Truck, Trailer and other vehicles (OV) (Motorbikes, Rickshaw, Tractor Trollies and Animal driven carts).

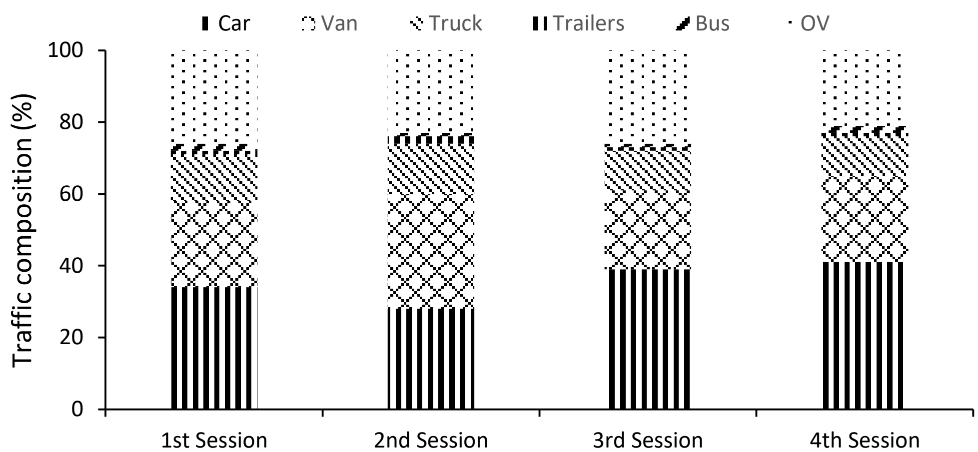

Figure 1: Classified traffic count for four sessions

Figure. 1 illustrates the traffic composition for four sessions. Presence of car and vans in the traffic stream was observed to be significant in a way that they together share about 60–65 percent of total traffic. Whereas proportion of truck and trailers were on lower side: around 10-15 and 1-2 percent respectively. It was also observed that substantial number of slow-moving vehicles like Rikshaw, Motorbikes and Tractor trollies were also present in traffic stream at the time of study.

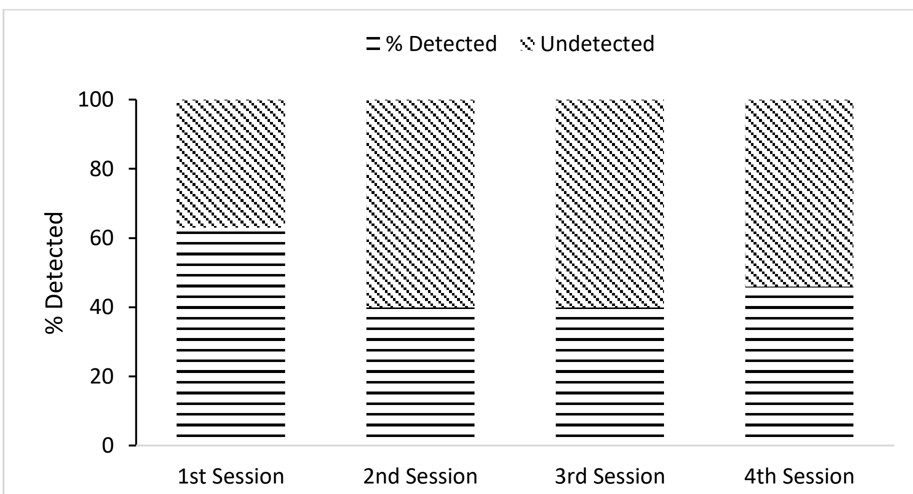

Figure 39: Illustration of license plate matching data

License Plate Matching Technique data was collected based on guidelines suggested by travel time data collection handbook (Turner et al., 1998). Every time a vehicle is entered or exit the study section its license plate number and time is noted in data sheets. These sheets were later evaluated to calculate the travel time (TT) based on time in (TI) and time out (TO) of each vehicle. Whereas speed of each category of vehicle was calculated using time-distance relationship (5). The total number of license plates matches were 259 that was well over the minimum requirement of 50 as recommended in previous literature studies and that too is extended over a period of two hours. Figure 2 show the percentage of total vehicles detected during the study period that ranges from 40-63 percent for four sessions. Although the free flow travel time in the section was 6 to 7 mins but there were several vehicles that entered the study section did not observed at the exit station even after an hour. This supported the fact that they dispersed in the study section and were using the facility as a collector or a local road.

In study section of 9 km posted speed limit for first 7.5 km was 100 kmph and 90 kmph for LTV and HTV respectively. Whereas, as the highway approaches urban area speed limit is reduced to 50 kmph for remaining 1.5 km of the study sections for all type of vehicles. So, the average speed for the section comes out as 85 kmph and 80 kmph. It is assumed that vehicles are moving with the posted speed so by using (5) the TT can be calculated as:

$$TT = \frac{s}{v} \qquad (5)$$

For 7.5 km

$$TT = \frac{7.5}{100} * 60\, mins$$

$$TT = 4\, mins\ 30\, sec$$

Whereas for 1.5 km

$$TT = \frac{1.5}{50} * 60$$

$$TT = 1 \min 48\, sec$$

So, travel time for the section is 6 mins and 18 sec for LTV. Similarly, Travel time for HTV should be 6 mins 48 sec based on the posted speed limit.

4. Results and Analysis

According to the methodology explained above the travel time data is collected for a specified length of road that was afterwards used to calculate the SMS (4). Speed limits are usually decided based on 85th percentile speed (Fitzpatrick, 2003) therefore 85th percentile speed and travel time is considered here for analysis. While considerable number of slow-moving vehicles were observed during the study period but as the study is focusing on assessing the mobility of through traffic movement, so speed and travel time analysis is not carried out for those vehicles. These are mostly localized traffic and they usually travel at very low speed with longer travel times. Classified speed and travel time data is summarized in Table 1.

Table 1: Classified Speed and Travel Time Data

Vehicle Type	SMS (kmph)	85th Percentile SMS	Aver. TT (mins)	Max TT (mins)	Min TT (mins)	85th Percentile	Variance (%)
Car	54.8	65.1	10.1	16.0	7.3	11.9	2.9
Van	53.6	61.9	10.4	22.0	6.5	12.0	4.0
Truck	40.7	53.4	14.6	27.7	9.0	19.8	24.6
Trailer	37.6	44.3	15.0	17.6	11.0	17.2	12.2
Bus	53.2	57.0	9.7	10.0	9.2	10.0	0.2

According to (Fitzpatrick, 2003) multilane divided highway passing through a urban area having level terrain (as in case of this study) have anticipated operating, posted and design speeds as 80-100 ,70-90 and 70-100 kmph respectively. But if we compare the speed values in Table 1 with the anticipated limits a significant reduction in speed can be witnessed for all the vehicle categories. Moreover, speed reduction can also be verified by equating the actual posted speed with the observed values in the field. These evaluations strengthen the hypothesis that the overall mobility of highway has been significantly reduced. The major speed reduction is observed in case of trailer whereas the least in case of cars. Trucks are also having variance of 24.6 % in speed values that can be backed by reason as those trucks that are not carrying any load were able to travel at high speed as compared to the loaded trucks.

4.1 Travel Time

Classified Travel time was calculated based on the data acquired from the license plate study. As expected longest average travel time is for trailers i.e. 15 mins with a maximum value of 17.6 mins. On the other hand, the buses have the shortest travel time of 9.7 mins with average speed as 53.2 kmph. In case of trucks average TT is 14.6 mins with a highest value of as high as 27.7.

Table 2: Free Flow Travel Time vs 85th Percentile Travel Time

Vehicle Type	85th Percentile TT (Mins)	Free Flow TT (Mins)	Delay (Mins)	Travel Rate (Mins/km)	Planning Time Index (PTI)
Car	11.9	6.3	5.6	1.32	1.88
Van	12.0		5.7	1.33	1.90
Truck	19.8	6.8	13	2.20	2.91
Trailer	17.2		10.4	1.91	2.52
Bus	10.0		3.2	1.11	1.47
Average	14.18	6.5	8.3	1.57	2.13

Table 2 shows comparison between free flow travel time and 85th percentile travel time. The delay computed for the corridor is 8.3 mins indicating that total travel time is more than double of the free flow travel. A similar research effort done by (Ter Huurne & Andersen, 2014) for major arterials of a town in south Africa found a delay of 3.37 mins on a weekday for the same time period as in this study. Whereas value as high as 12.79 for afternoon peak hour traffic were also observed during that research. Although peak hour analysis is not studied here in this paper, but it is highly possible that peak hour delay would be much higher than off-peak value of 8.3 mins. In case of individual vehicle type the highest delay is observed for trucks whereas the travel time of buses are closest to the free flow travel time. But as the buses are only 1% of the total stream, we cannot access the actual travel condition based on it. However, the travel time of car and van are significant here because they composed approximately 65 % of the total traffic. They were taking almost double of their free flow travel time i.e. approximately 12 mins instead of 6.3 mins to traverse the section. The trailers and trucks also have higher differential values of 10.4 and 13 mins respectively that can be due to vehicular characteristics or due to prevailing roadway conditions. Travel rate also confirms similar trends for each vehicle category.

Planning time index values are exhibited in Table 2. Based on PTI results it is revealed that travel time on the study section is 2.13 times more than free flow travel time which is a substantial increase for a category of facility type like multilane intercity highway. Another study conducted under similar conditions in India by (Bharti et al., 2018) observed values of 1.37 and 3.2 times more travel times for urban uninterrupted and interrupted flow urban corridor respectively. However, these values are for morning and afternoon peak periods so in case of this study 2.13 times more travel time that too in off-peak hours shows the severity of situation on the highway. Both the national highways have similarities like open accessibility and urban road side environment. (Mehran & Nakamura, 2007) found that a trip may take 30 % and 100 % more time in comparison to free flow travel time during the morning and afternoon peak periods respectively on Tokyo Expressway in Japan. The study was conducted in limited access environment with speed limit of same as in case of this study i.e. 100 km/h. So, this supported the fact that open accessibility is one of the major factors contributing to the reduction of highway mobility.

4.2 Speed Reduction Index

Speed reduction index reflects the level of congestion based on the difference between speed and free flow conditions. The baseline condition (FFS) for the analysis is identified as posted speed. However, in case of the selected road section there were two different speed limits as shown in table. So average speed is considered for the analysis i.e. 85kmph and 80kmph for LTV and HTV respectively.
The highest reduction in FFS occurs for trucks and trailers that needs to be further investigated as there could be several other roadway impedance factors that are impacting their speed. But it is quite evident that as the average speed of trucks and trailers reduce in a traffic stream it also impacts the speed of cars particularly when we are considering multilane divided highway having only two lanes in direction of travel. Furthermore, the FFS of cars and vans were also reduced to almost 25 % from which we can accomplish a fact that mobility of the highway was impacted since these two vehicle categories are almost 60 percent of the traffic stream.

Table 3: Weighted Speed Reduction Index for study section

Vehicle Type	Average Speed (Kmph)	Average FFS (Kmph)	Ratio	% Reduction in FFS	Speed Reduction Index	Weighted factor	Weighted Speed Reduction Index
Car	65.1	85	0.76	24	2.34	0.46	1.07
Van	61.9		0.72	28	2.71	0.34	0.92
Truck	53.4	80	0.66	34	3.33	0.16	0.53
Trailer	44.3		0.55	45	4.46	0.02	0.08
Bus	57.0		0.71	29	2.87	0.02	0.05
Total	-						2.65

Table 3 illustrate the application of speed reduction index. It was calculated individually for each vehicle and is then combined as weighted average to compute the overall congestion on the highway. The weighted is assigned based on the percentage of each category of vehicle in traffic stream. The overall speed reduction for the facility comes out as 2.65. While congestion usually occurs when the index exceeds 4 to 5 (Lomax et al., 1997) still it was apparent that the highway experiences considerable congestion during the study period. Moreover, the study section is a national highway facility offering intercity connectivity so even a congestion value of 2.65 is more for a facility having speed limit of as high as 100 kmph. Also, the selected study section has a suburban or a rural roadside development and data was collected when there was low to moderate traffic on the highway possibly this congestion value would be much more when the highway passes a pure urban environment during rush traffic hours.

5. Conclusions and Future Recommendations

This paper aimed to quantify the mobility of multilane highway section based on travel time and other congestion measures. Travel time reliability measures were also calculated, and results were compared with other similar studies conducted in developed and underdeveloped countries.

It is observed that the travel time for the LTV is almost double the free flow travel time whereas these values are even higher for HTV. At the same time high planning time index value shows the unreliability of the travel time on the highway. Similar is the situation with reduction in free flow speed where values range from 24 percent for cars to as high as 45 percent for the trailers. However, detailed data related to factors impacting travel speed are not available in the present study, therefore it is important that in future research those factors should be identified and quantified.

Speed reduction index for the facility did not confirm any severe condition of congestion as study section had a suburban roadside environment but it is most likely that congestion do occur in pure urban sections with high traffic volumes. This could be further investigated by taking data on urban section during peak traffic hour.

Another key finding of the study is that though it is a national highway that connects the major cities of country most vehicles are using the highway for the intracity or local movement that is a function of collector or local road.

Based on collected field data and comparison with other highways in the world it is observed that open accessibility is a major factor contributing to reducing the mobility of the highway. In the future research, data from other/more sections can be used to validate the results of this study and an aggregated model can be developed based on the factors impacting mobility.

References

Berry, D. S. (1952). *Evaluation of Techniques for Determining Over-All Travel Time.* Paper presented at the Highway Research Board Proceedings.

Bharti, A. K., Sekhar, C. R., & Chandra, S. (2018). Travel time reliability as a level of service measure for urban and inter-urban corridors in India. *CURRENT SCIENCE, 114*(9), 1913.

Fitzpatrick, K. (2003). *Design speed, operating speed, and posted speed practices* (Vol. 504): Transportation Research Board.

Hancock, M. W., & Wright, B. (2013). A Policy on Geometric Design of Highways and Streets. *The American Association of State Highway and Transportation Officials*.

Hummer, J. E., Robertson, H. D., & Nelson, D. (1994). Manual of TransportationEngineering Studies. *Institute of Transportation Engineering, by Prentice-Hall, inc, Englewood Cliffs, New Jersey*.

Lasley, P., Lomax, T. J., Eisele, W. L., & Schrank, D. L. (2014). Developing a total peak period travel time performance measure: An updated concept paper. *Transportation Research Record, 2420*(1), 15-22.

Lomax, T., Turner, S., Shunk, G., Levinson, H., Pratt, R., Bay, P., & Douglas, G. (1997). QUANTIFYING CONGESTION. VOLUME 2: USER'S GUIDE.

Manual, H. C. (2010). HCM2010. *Transportation Research Board, National Research Council, Washington, DC*.

McShane, W. R., & Roess, R. P. (1990). *Traffic engineering*.

Mehran, B., & Nakamura, H. (2007). *Performance Evaluation of Highway Segments Using Travel Time Based Performance Measures.* Paper presented at the Proceedings of the Eastern Asia Society for Transportation Studies Vol. 6 (The 7th International Conference of Eastern Asia Society for Transportation Studies, 2007).

Roess, R. P., Prassas, E. S., & McShane, W. R. (2011). Traffic engineering.

Ter Huurne, D., & Andersen, J. (2014). A quantitative measure of congestion in Stellenbosch using probe data.

Turner, S. M., Eisele, W. L., Benz, R. J., & Holdener, D. J. (1998). Travel time data collection handbook: United States. Federal Highway Administration.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan.

Performance Evaluation of MRF under Real and Artificial Ground Motions: A Study of Quetta Region Baluchistan

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Abstract

The seismic performance of moment frames under ground motion have been research interest for more than 3 decades. However, seismic design and compliant structures have always been of less concern in Pakistan. Most regions of the country are seismically active, where simple frame designed under gravity loads only, are insufficient to with stand ground motions. This study deals with behavior of concrete moment resisting frames (MRF) under real and synthetic ground motions in the seismic zone 4 of Pakistan Building Code 2007. The time history analysis of MRF is performed using 3D finite element model. The displacements and internal forces of MRF under real and synthetic ground motions are compared to check the seismic performance of MRF in seismic prone region.

Keyword
Real and Synthetic Ground Motions Comparison, Moment Resisting Frames (MRF), Time History Analysis.

Introduction

Most of the developing and underdeveloped parts of the world primarily make use of traditional masonry construction for residential purposes, because people in these countries are drawn towards making self-build dwellings of traditional practice and materials. The traditional construction practice is also motivated by the climatic condition of the country where the people need massive roofs and walls to protect the inside of residences from the extremity of outside temperature and high humidity. But these traditional practices lead to loss of human lives and economy of the country, to counter this the demand of structural design of buildings under seismic loads is becoming high day by day in all over the world, where seismicity is at its peak. However, seismic design is not of serious consideration for low rise buildings but due to rapid urbanization, there is a need for vertical construction for which seismic design plays a key role. Seismic criteria have been of less concern in the structural design among people and the structures constructed in seismically active regions are insufficient to withstand natural ground motions where seismic provisions are not considered. Safe seismic structures are built over a few decades after several seismic events i.e. 1933 Honshu Japan earthquake,1989 Lama Prieto earthquake,1994 Northridge earthquake and 1935 Quetta earthquake, as Quetta is the earthquake-prone region of Pakistan. The largest earthquake that occurred in this region on 30 may 1935, completely disfigured the city of Quetta, more than 60000 people were buried under the debris of the city, and Quetta building code was developed after 1935 earthquake in 1937 by the British government within the municipal limits of Quetta city. After the earthquake occurred in 1941, it was found that the buildings constructed in compliance with this code performed well(Naseer, Khan, Hussain, & Ali, 2010). Considering those several but seismic events, reinforced concrete special moment frame concepts were introduced in the U.S. starting around 1960(Blume, Newmark, & Corning, 1961) and essential till 1973 for high seismicity according to uniform building code, but the frames that were constructed considering only gravity loading in designing of SMRF did not perform well in 1994 Northridge earthquake, leading to precise requirements for proportioning and detailing these frames(Hooper)

Recent earthquakes in Pakistan demonstrated that the region is highly seismic. Masonry buildings constructed with stones, concrete blocks, and fired-clay bricks and concrete buildings were damaged during the 8 October 2005 Kashmir earthquake. Most of the buildings were observed to be nonengineered or semi-engineered. ((Bothara & Hiçyılmaz, 2008))

To check the seismic behavior of structure there are two types of analysis i.e. static and dynamic analysis which are based on building codes and real ground motions. As in the 1994 Northridge Earthquake, the frames did not perform well due to considering only gravity load and new provisions were needed. Similarly, when Pakistan Building code (1986) was developed by Federal Ministry of Housing and Works (MOHW) based on 1982 edition of Uniform Building Code as it lacked seismic provisions for concrete, steel and masonry buildings(Naseer et al., 2010), the frames did not perform well after 2005 Kashmir earthquake and there was a need to adopt new seismic provisions. These new provisions i.e., Pakistan Building Code (2007) are based on the 1997 edition of Uniform Building Code (1997).

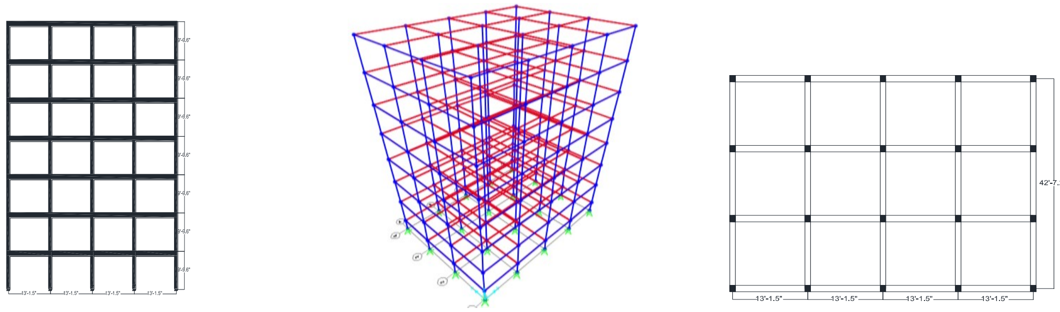
This research work presents the comparison of structural behavior under real and artificial ground motions. Ground motions generated synthetically with the help of design response spectrum which is generated according to the codes (UBC 1997) and scaled ground motions, that matched a suitable earthquake time history, applied on a frame and the results are discussed.

Finite Element Modeling

This study considers a midrise concrete structure for analysis. Such structures are seldom being seen in the study region as the regional code prohibits the height up to four-story. The frame selected for this study is a four-bay by three-bay seven-story Moment Resisting Frame, story height is 9.8' and the rectangular plane of 52’6” by 42’7.2”. the grade of concrete and steel used in the structure are M30 and Fe45 respectively. The frame is extracted from the “Earthquake Behavior of Buildings(Murty, Goswami, Vijayanarayanan, & Mehta, 2012).

Table 1: Input parameters

Sections	Dimensions
Column cross-section	11.8” ×15.7”
Beam cross-section	15.7”×15.7”
Slab thickness	5.9”
Dead load on beams from infill walls	209psf
Live load on slab	63psf
Modulus of elasticity of concrete	626400000psf
Compressive strength of grade M30 concrete	626563.02psf



(b) (a) (c)

Figure 1: 3D view (a), elevation (b), plan (c) of seven story frame

The foundation of the frame is considered as fix support, by restraining Moment and Forces in all directions. The internal reactions and displacements are calculated for dynamic loadings. The dynamic loadings include artificial and real ground motions. Linear behavior of the MRF is also studied.

Ground Motions

As the objective of the study, the structure has to be analyzed for both real and artificial ground motions. Artificial ground motions are generated using Seismoartif. As records of real ground motions are not available for the study region Quetta, a suitable earthquake has been selected and scaled using Seismomatch. The design spectrum is also generated, which helps in scaling and generating time-domain histories. Specifications for Quetta region are:

Table 2: Input parameters for generating Design Spectrum compatible with UBC-97

Parameter	Value
Zone factor	0.4
Soil profile	S_D
Na (source type=A, Distance=5km)	1.2
Nv (source type=A, Distance=5km)	1.6
Ca	0.44*1.2=0.529
Cv	0.64*1.6=1.024
T_s	$C_v/2.5C_a$=0.775
T_o	$0.2T_s$=0.156

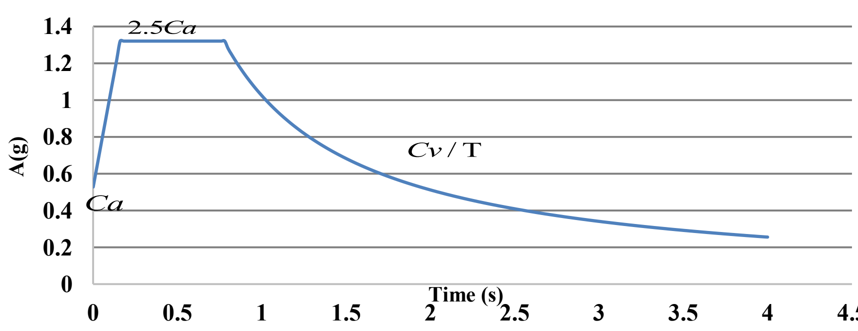


Figure 2: Design response spectrum.

Scaling of accelerogram

The real ground motion has been used to get a better idea of the structural performance during earthquakes. Properly selecting and scaling of real ground motion has done for this purpose. The scaling is done using a wavelet algorithm (Abrahamson, 1992)which is primarily based on the time-domain method (Lilhanand & Tseng, 1988). Earthquakes histories of ChiChi, Holistor, Imperial Valley and Northridge are used for scaling to best find the most relevant data. The scaling is done according to the local condition of Quetta following UBC97 for soil Type-SD (NEHRP classification), damping value of 5%, zoning factor of 0.4 and seismicity zone of 4.

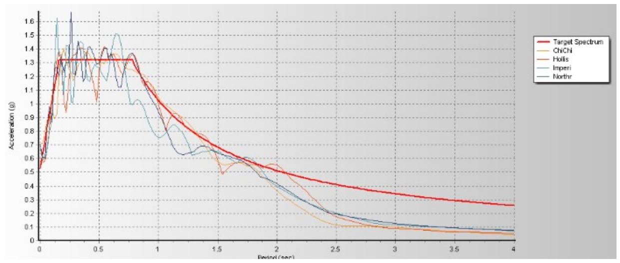


Figure 3: Response spectrum of different accelerograms.

The 1994 Northridge earthquake is selected for the analyses, as it best matches our required data with the average miss fit of 9.4%.

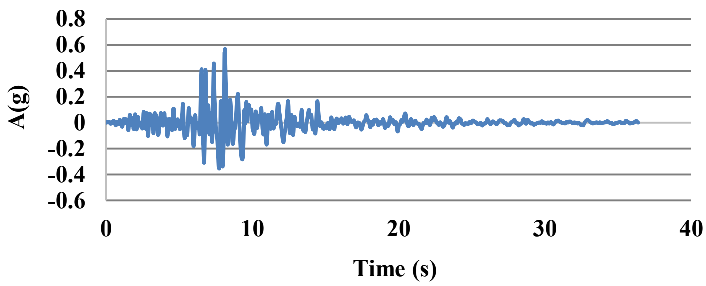


Figure 4: 1994 Northridge Earthquake original accelerogram

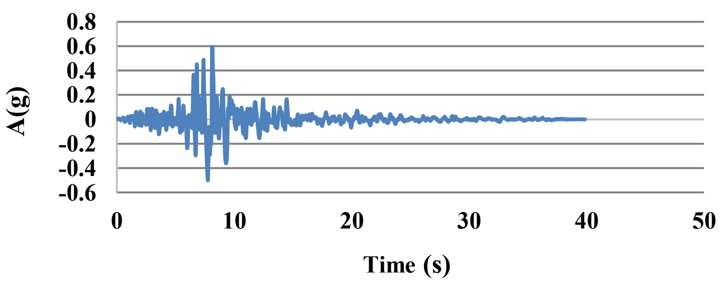


Figure 5: Scaled accelerogram for specified region.

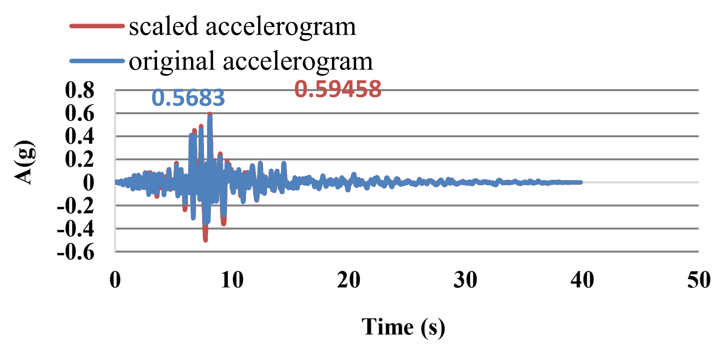


Figure 6: comparison of original to the scaled accelerogram

The PGA of the 1994 Northridge earthquake is 0.568g which is increased to 0.594g for the study area.

Synthetic Ground Motions

This study considers the effect of synthetic ground motions on the selected frame. The motions are generated following the target spectrum of UBC-97. The artificial accelerograms are generated by Power Spectral Density Function (PSDF). The PSDF is calculated from the velocity of the UBC97 design spectrum. Saragoni & Hart (Rodolfo Saragoni & Hart, 1973)envelope shape was selected to work with, having time at unitary intensity, $t_{1,}$ the value of intensity at last instant, t_{dur} and envelope duration is 4s, 0.05s. and 20s respectively. Seven artificial accelerograms are created having time-step 0.02 sec for a duration of 20s, with a damping ratio of 5%. Out of the seven, only one is selected for analysis. The means error for the selected time history is 6.92%.

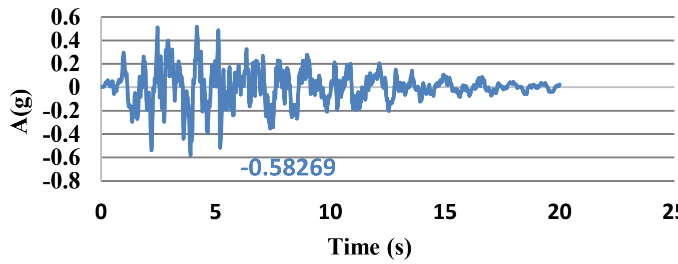


Figure 7: synthetic ground motion compatible with UBC97

The generated ground motion is having Peak Ground acceleration (PGA), Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD) of 0.583g, 74.4cm/s and 26.2m respectively.

Elastic response Spectrum

The Response Spectrum of the synthetic time history is also generated following the Newmark direct integration method(Newmark, 1959) with Beta is to 0.25 and Gamma 0.5.

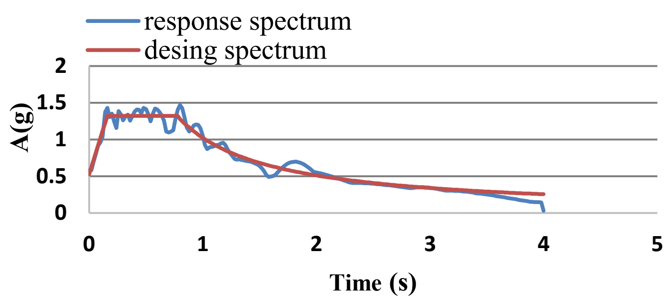


Figure 8: Response Spectrum

RESULTS

The frame is analyzed for combined (gravity plus dynamic) loadings on a finite element based software. To compare the behavior of artificial and scaled ground motions, the Frame is analyzed only at a few joints and elements namely as joint 757, 769 and 717 and member 1047. The reason for such few selections here is to have more clear vision of the study. The plot functions are generated only for 10 secs for both dynamic loadings.

Dynamic analysis

Joints and member's outputs based on Artificial Ground Motions (AGM):

Figure (9) shows the displacement of JOINT 757 in the direction of applied Earthquake. The maximum and minimum values of displacements are 25.24inch and 23.89inch respectively.

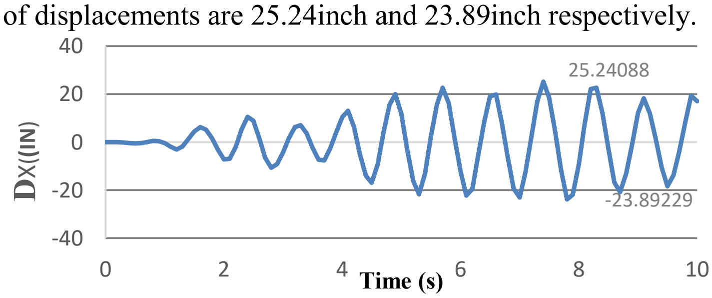


Figure 9: Joint output in X-direction

The displacement in Y-direction is relatively small that it can be ignore while comparing the MRFs under different ground motions. The reason for such small displacement in the given direction is due to the fact that no Ground Motions are applied in this direction. The maximum and minimum displacements are 5.17E-04inch and -5.62E-04inch respectively.

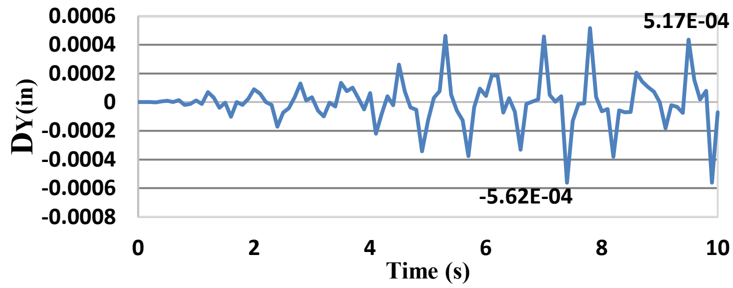


Figure 10: Joint output in Y-direction

Figure (11) shows the shear force values in member 1047. The maximum shear force in the member 1047 is 37.93643kips and minimum shear force is -35.83027kips.

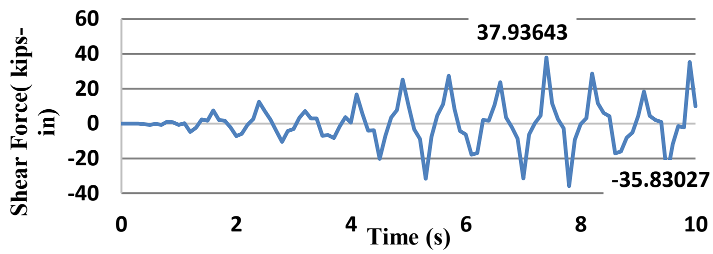


Figure 11: Shear force in member 1047

Figure (12) shows the values of bending moment at different intervals of time in the same member, with a maximum value of 158.49801 kips-in and minimum value of -157.85601 kips-in

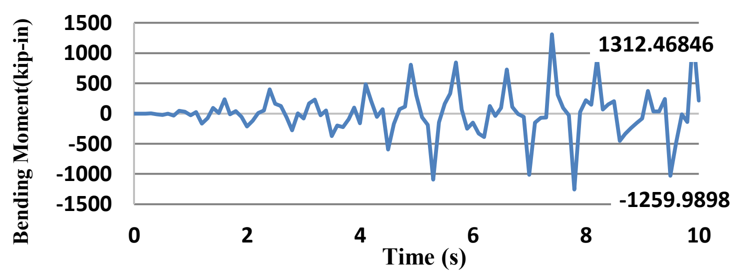


Figure 12: Bending moment in member 10

The maximum and minimum values for displacements, shear forces and bending moments of the selected joints and member has illustrated in table (3).**Table 3: internal forces**

Position	Outputs	Maximum	Minimum
Joint 717	X displacement (in)	21.219054	20.119899
	Y displacement (in)	0.000681	0.000656
	Z displacement (in)	0.327642	0.550844
Joint 757	X displacement (in)	25.241448	23.891726
	Y displacement (in)	0.000981	0.000981
	Z displacement (in)	0.335253	0.582382
Joint 769	X displacement (in)	25.241817	23.893176
	Y displacement (in)	0.00013	0.00013
	Z displacement (in)	0.291704	0.291704
Member 1047	Shear force (kips)	37.93643	-35.83027
	Bending moment(kip-in)	1312.46846	-35.83027

Joints and elements outputs based on scaled Ground Motions:

The same frame was evaluated for real ground motions scaled according to the locality of the study region. The outputs are generated for the same joints and members as for previous artificial ground motions loadings. The evaluation is done in terms of displacements, shear forces and bending moments. Figure (13) shows the displacements in global X-direction under scaled ground motions. The positive and negative displacement in the direction of loading is 18.74509inch and 19.81194inch respectively

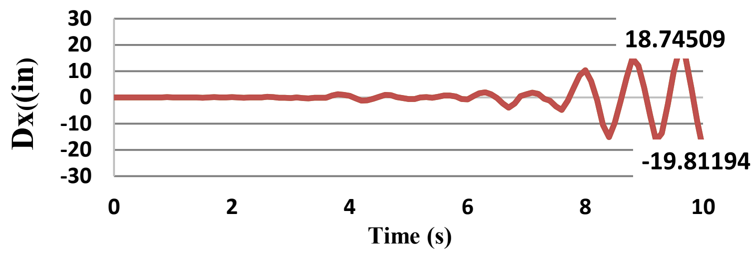


Figure 13: X displacement at joint 757

Similarly, the displacement in Y-direction is also calculated for scaled ground motions. The graph shows a maximum and minimum displacement in this directions are 2.60E-04inch and -2.79E-04inch respectively.

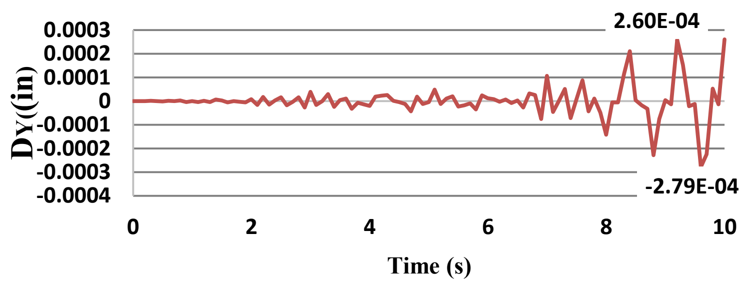


Figure 14: Y displacement at joint 757

Detailed outputs of the selected joints and member has been shown in the table (4) where maximum and minimum displacements of 18.745663 and -19.811373inches can be seen on JOINT 757. The maximum displacement obtained here is in the direction of applied dynamic forces

Table 4: internal forces

Position	Outputs	Maximum	Minimum
Joint 717	X displacement (in)	15.94499	-17.022719
	Y displacement (in)	0.000439	-0.000407
	Z displacement (in)	0.217897	-0.473754
Joint 757	X displacement (in)	18.745663	-19.811373
	Y displacement (in)	-0.000157	-0.000697
	Z displacement (in)	0.21856	-0.499083
Joint 769	X displacement (in)	18.745482	-19.812288
	Y displacement (in)	0.00013	0.00013
	Z displacement (in)	-0.291704	-0.291704
Member 1047	Shear force (kips)	22.03481	-21.39567
	Bending moment(kip-in)	638.25985	638.25985

Comparative study:
The results obtained from FEM based analysis can easily be interpreted. To have a clear vison of the study the results are compared in the form of tables and graphs.

Figure (15) shows the comparisons of displacements in X-direction. The graph is plotted between the displacements of JOINT 757 due artificial and scaled ground motions. The peak values for displacement remain the same as that of previously calculated.
Similarly, a graph has also been plotted for the same kind of loadings in Y-direction. Figure (16) shows the comparisons of translation of JOINT 757 in direction perpendicular to the direction of applied loadings,

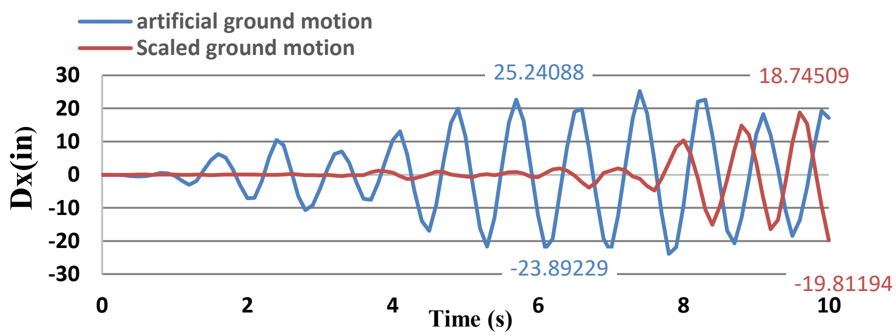


Figure 15: Comparisons of displacements in X-direction JOINT 7

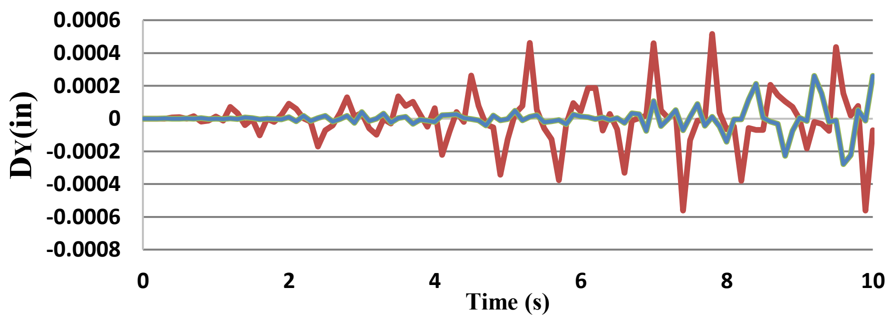


Figure 16: Comparisons of displacements in Y-direction on JOINT 75

Figure (17) shows the comparisons of shear force in member 1047. The graph shows that the shear force due to artificial ground motion are much greater than that of shear force due to scaled ground motions.

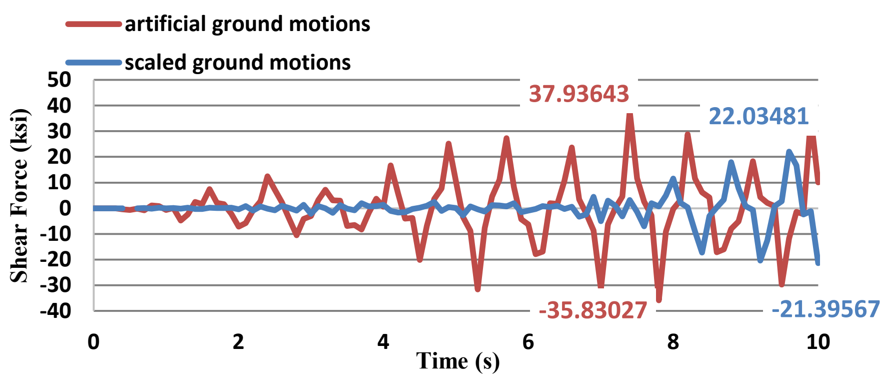


Figure 17: Comparison of shear forces in member 1047

Figure (18) shows the comparisons of bending moments in member 1047, obtained separately for artificial and scaled ground motion. Here the result due to artificial ground motion are much greater than that of scaled ground motions.

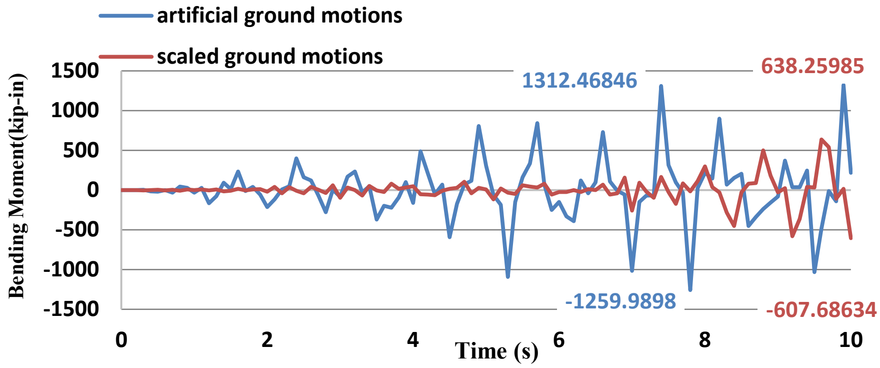


Figure 18: Comparison of bending moment in member 1047

Table (5) shows complete comparisons of joint displacements, member's shear force and bending moment. The table consists of maximum displacements and internal forces for both the ground motion.

Table 5: comparison of maximum values for artificial vs scaled ground motions

Position	Outputs	Artificial ground motions	Scaled ground motions
Joint 717	X displacement (in)	21.219054	15.94499
	Y displacement (in)	0.000681	0.000439
	Z displacement (in)	0.327642	0.217897
Joint 757	X displacement (in)	25.241448	18.745663
	Y displacement (in)	0.000981	-0.000157
	Z displacement (in)	0.335253	0.21856
Joint 769	X displacement (in)	25.241817	18.745482
	Y displacement (in)	0.00013	0.00013
	Z displacement (in)	0.291704	-0.291704
Member 1047	Shear force (kips)	37.93643	22.03481
	Bending moment(kip-in)	1312.46846	638.25985

The overall result can be concluded in the form of bar chart. Figure (19) shows the evaluation of displacement and internal forces. The shift of JOINT 717, JOINT 757 and JOINT 769 due to artificial ground motion is considerably greater than the shift due scaled ground motion. The shear force in the member 1047 calculated for artificial ground motions is almost double the size of shear force induced by scaled ground motions. Bending moment in the same member due artificial ground motions is more than twice of that due scaled ground motion

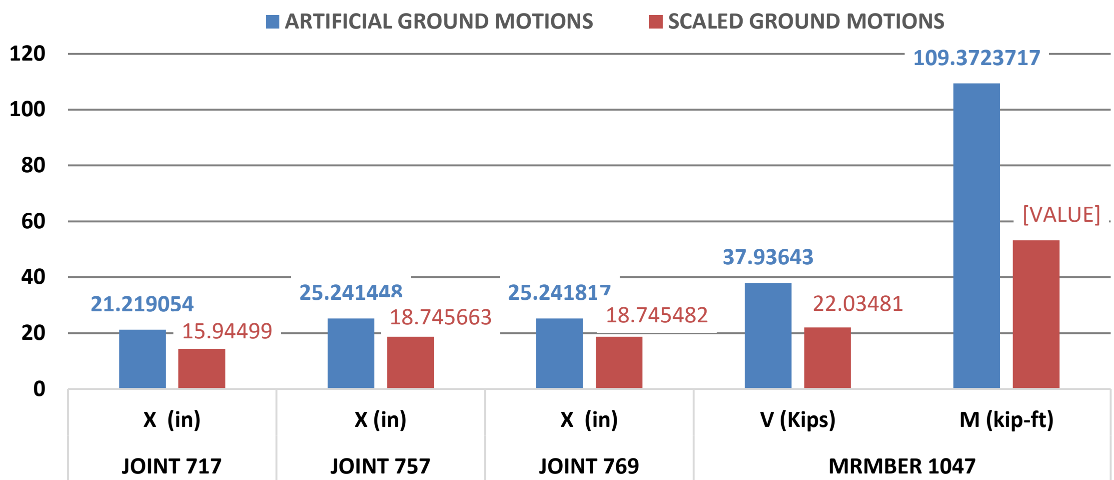


Figure 19: comparison of displacement, shear force and bending moments induced by artificial and scaled ground motions

Conclusion:

This study has concluded that artificial ground motions yields higher results in every aspect. To be more conservative about the structure, artificial ground motions can be used as dynamic inputs. While if economy is also an element of consideration scaled ground motion has to be used for the analysis and design of Moment Resisting frames in the region of Quetta.

References

Abrahamson, N. (1992). Non-stationary spectral matching. *Seismological research letters, 63*(1), 30.

Blume, J. A., Newmark, N. M., & Corning, L. H. (1961). *Design of multistory reinforced concrete buildings for earthquake motions* (Vol. 4): Portland Cement Association Chicago.

Bothara, J. K., & Hiçyılmaz, K. (2008). General observations of building behaviour during the 8th October 2005 Pakistan earthquake. *Bulletin of the New Zealand society for earthquake engineering, 41*(4), 209-233.

Hooper, J. P. M. J. D. Seismic Design of Reinforced Concrete Special Moment Frames a Guide for Practicing Engineers, SECOND EDITION.

Lilhanand, K., & Tseng, W. S. (1988). *Development and application of realistic earthquake time histories compatible with multiple-damping design spectra.* Paper presented at the Proceedings of the 9th world conference on earthquake engineering.

Murty, C., Goswami, R., Vijayanarayanan, A., & Mehta, V. (2012). Earthquake behavior of buildings. *Gujarat State Disaster Management Authority, Gandhinagar*, 53-79.

Naseer, A., Khan, A. N., Hussain, Z., & Ali, Q. (2010). Observed seismic behavior of buildings in northern Pakistan during the 2005 Kashmir earthquake. *Earthquake Spectra, 26*(2), 425-449.

Newmark, N. M. (1959). *A method of computation for structural dynamics*.

Rodolfo Saragoni, G., & Hart, G. C. (1973). Simulation of artificial earthquakes. *Earthquake Engineering & Structural Dynamics, 2*(3), 249-267.

11th International Civil Engineering Conference (ICEC-2020)
"Integrating Innovation & Sustainability in Civil Engineering"
March 13-14, 2020, Karachi, Pakistan

Review on Code-based Seismic Analysis and Design Procedures of Elevated Water Tower

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Abstract
Elevated Water Tower (EWT) at certain height from ground level is aimed to supply water in a populated area and to facilitate in emergency conditions. Earthquakes are one of the major natural disasters. Water towers are one of the critical structures which can be severely affected by earthquakes. There is no single document based on which a EWT can be completely analyzed and designed against seismic loading. The specific goal of this study is to review seismic analysis and design procedures of elevated water tower available in different codes. ACI 371R and ASCE 7-05 have certain limitations due to which these cannot be used strictly for Pakistan due to non-availability of site specific acceleration amplification. The paper discusses the basis of damages caused to EWT during earthquakes, shapes of EWT, analysis and design methods, applicable software and different analysis parameters of EWT. The emphasis of this work is to formulate guideline procedure for seismic analysis and design of EWT along with the feasible software to be used in Pakistan region so as to have design in an efficient way.

Keywords
Water Tank, Elevated Water Tank and Water Tower.

1. Introduction

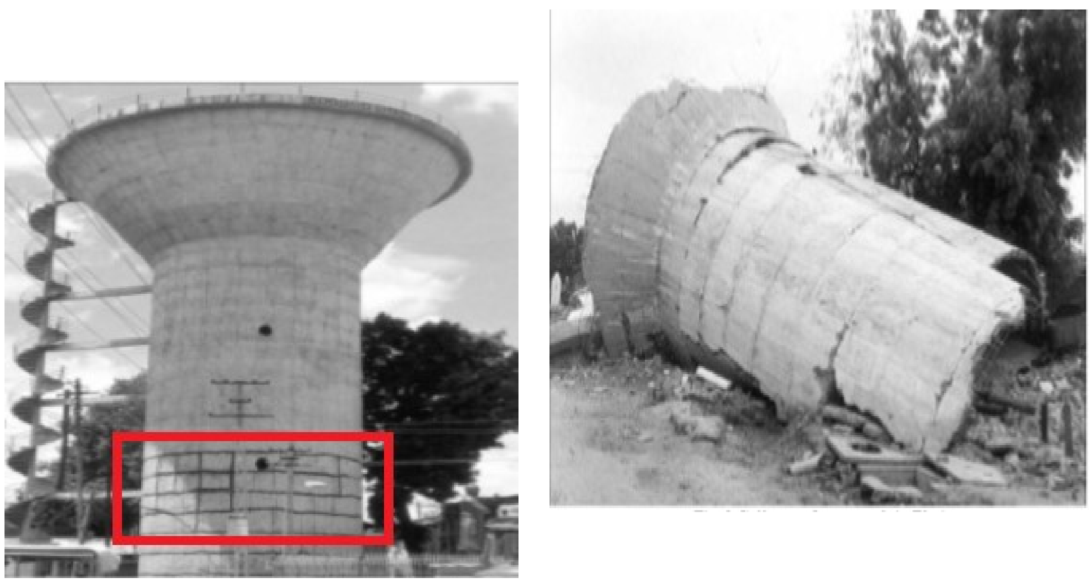
A water tower is an elevated structure which supports a water tank installed at a specific height that is adequate to pressurize water supply system for the distribution of drinking water and to facilitate during emergencies such as fire accidents. The word "standpipe" is used to refer to a water-tower in some cases. Water towers sometimes work in connection with the surface or underground reservoirs, which store the treated water close to the nearby vicinity for where it will be used. Some types of water towers may only store water for fire accidents and industrial uses and not for the public and domestic purposes. While there have been various forms of use of heavy tanks since ancient times, in the mid-19th century the modern use of the water towers developed for the pressurized public water systems, as steam pumping became more common and better pipes were developed, which were able to handle higher pressures. In the UK, standpipes consisted of large, open, n-shape tubes used for pressurized relief and for a steady rising of steam-powered hydraulic motors which appeared to create a pulsing stream and demand a constant pressure for pressurized water delivery.

Standpipes also provided an easy fixed place for measuring rates of flow. In decorative masons or wooden structures, designers typically enclosed the riser pipes. By the late 19th century, storage tanks were built to meet the growing demands of growing towns. The water towers can be surrounded by fancy brickwork, a big riveting, or they can be painted simply. Many civic towers feature the name of the city displayed as a navigational aid for variations and drivers in large letters on the building. The lighting can sometimes be amusing. A case in point are water towers, marked HOT and COLD, which are installed side by side. The height of the tower brings the water supply network stress and can be applied to it by means of a generator. The tank size and pipe diameter ensure and control the rate of flow. It is costly to rely on a pump to provide pressure; for a differing requirement, the pump should be designed to meet high demands.

The purpose of this paper is to review the seismic analysis and design of EWT on the basis of damages caused to EWT by earthquakes. Second aspect discussed in the paper is different shapes of EWT. Design methods and applicable software to analyze and design are also part of this paper. Paper also discusses different parameters used in analysis and design of EWT. The outcome of this work is to guide procedure for analysis and design of EWT along with the feasible software to be used in Pakistan region so as to have design in an efficient way.

2. Damages Caused to Elevated Water Towers by Earthquake

Natural hazard imposes a lot of effect on Structures like EWT. These disasters have very vast impact on economy and environment. Water towers are particularly vulnerable to natural event like earthquakes. This vulnerability can cause loss of valuable resources. So, water tower needs to be considered as a special structure whose safety is important than other structures (Oscar et al. 2019). The utility of the use of nonlinear fluid / structure interactions to monitor the seismic reaction to near-field earthquake systems is explored in this article. For this reason, the SDOF System is assumed to be fitted with a circular cylindrical liquid tank. In terms of its basic mode of motion, the design is an idealized version of the multi-degree principle of independence (Waezi et al. 2019).



(a) Flexure cracks in staging (b) Collapse of Water Tower

Figure 1: Failures of EWT (Pole and Khedikar 2017).

Throughout recent seismic excitations, fluid storage tanks above ground have suffered severe damage. In general, the violent movement of fluid in partially filled tanks triggered by earthquakes may result in severe sloshing loads on the hydraulic structure, resulting in the tanks ' regional yield, buckling, and irreparable failure (Cho et al. 2017).In water supply networks, high water tanks are important structures. While disasters, they must remain operational for critical purposes. Throughout major earthquakes, the seismic response of these inverted pendulum type structures have been recorded. Miscellaneous layout in structural member, low overall toughness and moldable capability, torsional amplification of seismic activity, and the impact of soil-structure interaction are illustrated in previous studies among several explanations to explain failures and harm in past earthquakes (Masoudi et al. 2012).

It is well established that water towers are generally exposed to dual direction shaking or bi directional shaking during seismic activity. Capacity assessment theories, though, were based primarily on uniaxial experimental findings without taking parallel behavior into account in two main directions (Aparna et al. 2014). During the dynamic analysis of the structures, it is mostly assumed that the structure will respond depending on the magnitude of the earthquake and its peak ground acceleration. However, studies show that the seismic hazard is dependent on the properties of the specific earthquake's acceleration time history. Research by researchers Anderson and Bertero have shown that properties of earthquake ground movement such as quality, length, velocity, displacement and incremental deflections may have critical effects on elevated water towers that have reaction relative to maximum ground acceleration (Waghmare et al. 2015).

3. Shapes of Elevated Water Tower

The basic purpose of the water tower is to retain and hold the fluid at a specific height which creates an adequate pressure for the distribution and even supply of water for domestic and industrial purpose by using pipes. Most often water towers receives their supply from deep well, dam, spring, stream or river by using a network of pumps which may be operated electrically, mechanically or chemically. There are many various shapes used for the constructed of water towers. The commonly ones used are circular, square, rectangular, triangular or even H shaped (Salajka et al. 2014). Latest construction practices includes the construction of RC mushroom shaped water tower which is more economical compared to other shapes and also offers more fluid holding capacity.

WT (b) Spherical Water Tower

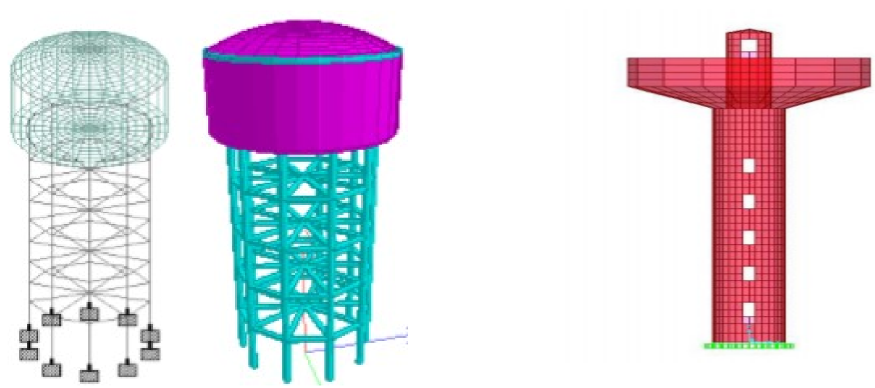
Figure 2: Different Shapes of EWT (*Water tower.Wikipedia.com.Web.11 Oct.2019).*

Various shapes that are used for the water towers such as circular, rectangular etc. With the passage of time, these are becoming outdated and being neglected in new construction practices as they are relatively costly to build and have less storage capacity compared to mushroom shaped EWT. Mushroom shaped EWT is the latest shape followed around the globe as it has more fluid holding capacity compared to other shapes. The elevated tower support system is divided mainly into two types namely frame staging and shaft staging. The distinctive structural characteristics of these two support systems indicate that these structures may not have the same damage patterns and failure mechanisms as confirmed in previous earthquake surveys. Therefore, the actions and seismic efficiency of tanks backed by frame and shaft must be assessed separately (Mostafa et al. 2012).

Elevated water tanks of reinforced concrete (RC) are among the most seismically prone lifeline structures built in the first half of the 20th century. This is attributable to the slender and tall structure and structural design of their level, where masses are most concentrated on top. In comparison, they often represent the revolutionary R / C technology applications, which are characterized by relatively poor architectural detail and mechanical properties of concrete and stainless steel. Their significant role and sophistry in the development of reinforced concrete architecture in the urban and suburban environment prompted the inclusion in the list of multi-countries heritage sites that were often located in medium-and high seismic areas around the globe of older high-altitude water towers. Based on applying low-impact reconstruction solutions that respect their recognized value in architecture and engineering, these systems currently require a seismic retrofit solution. A famous example, (Tranzeni et al. 2018).

4. Design Methods and Applicable Software's

Water tanks are the water storage vessels. Elevated water tanks are designed to provide the necessary head so that the water flows under the influence of gravity, the water tank construction method is as old as civilized people. The plan of water tanks has a high priority as it provides drinking water to the small population living in towns and villages for the vast population from major metropolitan cities. Overhead water towers and storage reservoirs are used to store petroleum or water. The force analysis of the tank remains same despite of the chemical nature of the product. All tanks are designed to avoid leakage as crack-free structures (Ahuja and dashore 2019).



(a) STAAD PRO model (b) SAP model

Figure 3: 3D Models of EWT in different software.

Tank size R, tank height L, tank wall spreads hs, and tank height H are the geometrical parameters. The concrete surface size is the same as the ship length. The principles of the fluid structure are that the water in the reservoir is idealized as compressible, vicious or rotating. The tank wall is related to convective as well as impulsive volumes by analogous spring kc and ki, whose values depend on the characteristics of the tank wall and the weight and volume of the water. The damping constants are convective and impulsive, and are the assumed damping values. (Naresh 2019). FEM is the best method for precise analysis of EWT (Chawla et al. 2020)

The thesis analyzes sloshing processes and CFD (compute fluid dynamics) with finite element stress analysis (FEA), which is used to estimate sloshing wave strength, convective mode rate, wall tension and slashing results at anchorages. The interaction between the fluids (water and air) and the tank wall shall be modeled, so that the relation is full or one direction. The results of the FSI framework were contrasted with those of simple mechanical systems in design manuals. This includes the simulation of the fluid structure (FSI) partnership in partially filled tanks (Nicolici et al. 2013).

The purpose of the study is to check the quality of higher seismic tanks. This research incorporates the finite element (FE) method to analyze the seismic reaction of the liquid-filled tank. The fluid domain is modeled with displacement-dependent fluid components. Recent and modal testing is carried out on a large tank. Specific impulsive and convective reaction components are obtained by means of the FE system. Further, the effect of tank wall flexibility and water-free surface sloshing is in the FE analysis (Moslemi et al. 2011).
Two different model configurations are investigated in three-dimensional space associated with shallow and tall tanks. A rectangular tank model's design display. The first step in a finite element analysis is to calculate the possible error by choosing a mathematical model to reflect the item being studied. The numerical goal for this experiment was selected as the maximum pressure at the bottom of the rigid tank using derived formulas. Prior to performing the review of time history, the basic phases of impulsive and convective actions and their associated mass ratios were determined using both finite element Analytical method for models of Elevated water tower. In accordance with ACI 350.3-06, fundamental analytical frequencies and related mass ratios are obtained (Kianoush et al. 2012).

5. Different Parameters in Elevated Water Tower

In the event of a major earthquake, it is possible to develop buildings that do minimal harm, but linear elastic analysis is not functional. The vastness of buildings with linear elastic structure is calculated by plurality of laws, for example the NEHRP (FEMA 1997), the UBC (1997) and the Iranian earthquake (BHRC 2010). The nonlinear behavior of the structural materials intrusion of the structure into the plastic area converts the reaction parameter (R) to actual non-elastic force with the earthquake intensity from the linear analysis. However, the value of R can differ proportionally due to the structural framework used in the model.

The multiple base motion effect on hydrodynamic pressure, displacement of the tank and the problem of elevation of the liquid layer in the elevated water tank are known as a question of Liquid-Structure-Soil interaction. Where the interaction between soil and structure induces rocking movement and the interaction between liquid structures causes the water tank. The action is hydrodynamic. According to a comprehensive study on the behavior of high water tanks of steel under simple rocking, tanks of liquid with horizontal and vertical earthquake agitation together with rocking motions (Adiyanto et al. 2019). The capabilities of liquid-filled water tanks correlated with earthquake slotting are important considerations. Due to potential negative economic and environmental impacts of turbine failure and liquid spillage in the surrounding area, seismic safety of an elevated water turret is of considerable concern. Research has been carried out to improve the seismic conduct of water towers in order to improve their safety (Terenzi et al. 2018). A comparative analysis of seven earthquake-accelerating information to determine the impact of the ground shape on the system conduct the conditions.

Two standard methods are introduced to design the relationship between the system and the surface, namely the direct method and the substructure method. The structure and the near-field soil surrounding the foundation were modeled for the direct method by the method of finite elements, the method of boundary elements as well as the method of coupling finite-boundary elements, while the semi-infinite element or infinite element is used to simulate the far-field soil. The coupling process is split into two subsystems for the substructure method: the subsystem of the superstructure and the subsystem of the soil-foundation. The soil is generally considered to be a homogeneous half-space. This is the complex form of impedance. Usually used in soil-structure interaction analysis, which describes the ratio between the exerted excitation force and the resulting steady-state displacement (or rotation) relationship, whereas difficulties in applying frequency-dependent impedance function to frequency domain analysis may be encountered. The frequency-independent LPMs are therefore suggested to reflect the complex rigidity of the interface between the soil and the base (Wenling et al. 2018).

6. Guideline procedure for analysis and design in Pakistan region

Seismic analysis and design of EWT considers lateral loads due to ground motions during seismic activity. There are mainly two types of analysis i.e. dynamic and equivalent static force procedure. This work discusses the equivalent static force procedure method. Keeping in view the importance of EWT sloshing effect must be considered during analysis in addition to the seismic lateral loads considered. Sloshing effect represents the oscillation of water surface due to ground movement due to earthquake. Stored water in EWT can be divided into two parts convective component and impulsive component. Convective component is upper part of oscillating water and impulsive is lower part of water.

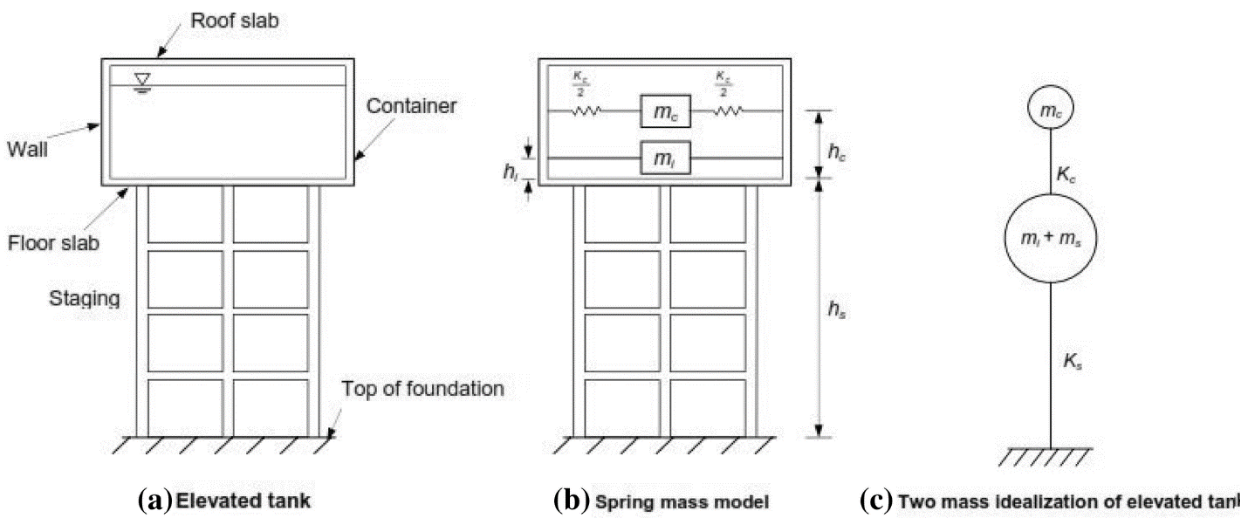


Figure 4: Simplified model of EWT for dynamic analysis, (Babu et al. 2019).

These two components and tank structure creates three structures which means three degree of freedom to be considered. Some researchers also suggested to use two degree of freedom but in equivalent static force procedure seismic mass and mass of sloshing water is considered. Equivalent static force procedure is in ACI 371R and ASCE 7-05 but these codes suggests site specific acceleration amplification factor which make it difficult to use in Pakistan region. Hence equivalent static force procedure method's concept is taken from ACI 371R. 5.1.2.4.1, G in 5.1.2.2 and 5.1.2.9 of ACI 371 are used for vertical and horizontal eccentricities of sloshing. Effective seismic weight should include dead load of tank, convective and impulsive weight of water, 25% of live load and 20% of snow load where roof snow load exceeds 30 Psf. Weight of convective and impulsive component should be calculated using ACI 350.1. Rest procedure could be adopted same as used for buildings with exceptions like using response modification factor as 2.2 and importance factor as 1.25. If other parameters are governing they should also be considered along with load combinations as per ACI 371R.

7. Conclusions

The overhead water tanks that may be used as utility holding storage tanks, as a buffer tank for water supply schemes and for refilling the tanks for several uses can be referred to as the water storage system constructed above the surface. The advantages of reinforced concrete water towers do not affect climatic changes, they are leak-proof, more rigid and are suitable for every shape. Such water towers are therefore of tremendous significance. Therefore, by taking structural protection into account, it is necessary to model the water tower correctly. For analysis and design in Pakistan region concept of equivalent static force procedure method should be adopted according to ACI 371R along with ACI 350 and ACI 370.

Acknowledgement

Authors would like to thank every person who helped during this study especially Sarosh Jameel and Mustaneer Ahmed Satti. The careful review and constructive suggestions by the anonymous reviewers are gratefully acknowledged.

References

Adiyanto, M. I., Damit, A. A. A., Yaakup, S. N., Mustapha, S. A. H. S., & Chin, S. C. (2019, February). Effect of column size on the seismic capacity of elevated reinforced concrete water tank. In IOP Conference Series: Earth and Environmental Science (Vol. 244, No. 1, p. 012010). IOP Publishing.

Ahuja, S., & Dashore, S. (2019). Implementation of Water Tank Design using Staad Pro Software. structure, 1, 5.

Babu, M. R., Singh, M. R., & Singh, L. (2019). Seismic Design Recommendations For Elevated Water Tanks.

Chawla, S., Jamle, S., & Meshram, K. (2020). A Review on Economical Design of Intz Water Tank as per IS-875-III, for Wind Speed in India. *International Journal of Advanced Engineering Research and Science*, *7*(1).

Chaduvula, U., Patel, D., & Gopalakrishnan, N. (2013). Fluid-structure-soil interaction effects on seismic behaviour of elevated water tanks. Procedia Engineering, 51, 84-91.

Cho, I. H., Choi, J. S., & Kim, M. H. (2017). Sloshing reduction in a swaying rectangular tank by an horizontal porous baffle. Ocean Engineering, 138, 23-34.

Ghosh, A. D., Bhattacharyya, S., & Roy, A. (2013). On the seismic performance of elevated water tanks and their control using TLDs. In Key Engineering Materials (Vol. 569, pp. 270-277). Trans Tech Publications.

Masoudi, M., Eshghi, S., & Ghafory-Ashtiany, M. (2012). Evaluation of response modification factor (R) of elevated concrete tanks. Engineering Structures, 39, 199-209.

Meng, X., Li, X., Xu, X., Zhang, J., Zhou, W., & Zhou, D. (2019). Earthquake Response of Cylindrical Storage Tanks on an Elastic Soil. Journal of Vibration Engineering & Technologies, 1-12.

Moslemi, M., Kianoush, M. R., & Pogorzelski, W. (2011). Seismic response of liquid-filled elevated tanks. Engineering structures, 33(6), 2074-2084.

Naresh, K. (2019). Seismic Analysis of Over Head INTZE Water Tank Subjected to Sloshing Effect.

Nicolici, S., & Bilegan, R. M. (2013). Fluid structure interaction modeling of liquid sloshing phenomena in flexible tanks. Nuclear Engineering and design, 258, 51-56.

Pole, S. M., & Khedikar, A. (2017). Seismic Behaviour of Elevated Water Tank under Different Staging Pattern. International Journal of Engineering and Management Research (IJEMR), 7(3), 199-205.

Terenzi, G., & Rossi, E. (2018). Seismic analysis and retrofit of the oldest R/C elevated water tank in Florence. Bulletin of Earthquake Engineering, 16(7), 3081-3102.

Waezi, Z., Attari, N. K., & Rofooei, F. R. (2019). Controlling the seismic response of structures under near-field earthquakes with fluid/structure interaction of cylindrical liquid tanks. European Journal of Environmental and Civil Engineering, 1-24.

Waghmare, M. V., Madhekar, S. N., & Matsagar, V. (2015). Behaviour of Elevated Water Storage Tanks Under Seismic Events. In Advances in Structural Engineering (pp. 1167-1176). Springer, New Delhi.

Waghmare, M. V., Madhekar, S. N., & Matsagar, V. (2015). Behaviour of Elevated Water Storage Tanks Under Seismic Events. In Advances in Structural Engineering (pp. 1167-1176). Springer, New Delhi.

Zohra, H. F., Mahmouda, B., & Luc, D. (2012). Vulnerability assessment of water supply network. Energy Procedia, 18, 772-783

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List of Papers Presented in 10th International Civil Engineering Conference (ICEC-2019) held on 23 & 24 February, 2019

"How Latest Technological Advancements are Transforming the Civil and Structural Engineering Profession "
keynote address by Dr. Naveed Anwar, Vice-President, Knowledge Transfer Asia Institute of Technology, Bangkok, Thailand on which was very well appreciated by the participants.
"Mechanical Performance and Durability of Recycled Aggregates in Ordinary and Self-Compacting Concrete"
Keynote Address in the 1st Technical by Prof. Said Kenai
Effect of methylcellulose on different properties of fresh and hardened concrete
- Zeeshan Ullah, Husnain Tariq But, Muhammad Saad, Muhammad Salman, Mustansar Hussain
Utilization of rubber powder of waste tyres in foam concrete
- Imtiaz Ali Bhatti, Zahid Hussain Khaskhelli, Mouzzam Ali Lohar, Seengar Ali Mehrani
Investigation on selected properties of concrete blended with maize COB ash
- Zubair Hussain Shaikh, Aneel Kumar, Manthar Ali Kerio, Naraindas Bheel, Ali Aizaz Dayo, Abdul Wahab Abro
Halcrow on SD BI M and its Industry Application
- Invited Talk by Engr. Ghulam Mujtaba Shaikh
Comparison of physical attributes of real time project using BIM - A case study
- Abdul Hannan Qureshi, Syed Wajhi U. H. Naqvi, Abdul Qadeer, Muhammad Kamran, Talha Bin Tahir
Hazard identification in construction safety: A visualization-based approach
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- Umair Imran, Muhammad Bilal Khurshid,, Muhammad Jawed Iqbal
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- Invited Talk by Engr. Ashar H. Lodi, Director Transportation, Exponent Engineers
Multi-criteria Decision Making in Urban System Traffic Evaluation
- Malik Kamran Shakir, Dr Muhammad Bilal, Dr Nadeem Anwar Qureshi, Dr Arshad Hussain, Muhammad Adeel
Potential of Intergrated Bus Rapid Transit (BRT) System in Motorcycle Dominant Cities
- Syed Fazal Abbas Baqueri, Muhammad Adnan,Engr. Prof. Dr. Mir Shabbr Ali
International Drivers' Comprehension of Traffic Control Devices-A Case Study for Foreign Drivers living in South Korea
- Sarang Jukhio, Jin-Tae kim
Earthquake Engineering Design Practice in Pakistan
- Invited Talk by Engr. Abul Khair Masroor, EA Counsultant Pvt Limited
Codes comparison for the seismic response of SMRF A case study of Quetta Balochistan
- Naik Muhammad, Muddassir Ahmed Khan, Nisar Ahmed, Muhammad Idress, Shafiullah, Aimal khan, Danish Haider, Muhammad Habib, Zafar Baloch, Saeed Ullah Jan Mandokhail
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- Youcef Mehani, Abderrahmane Kibboua, Benazouz Chikh, Mustapha Remki
Dynamic response analysis of submerged floating tunnel under waves and earthquakes
- Naik Muhammad, Zafar Baloch, Muhammad Habib, Saeed Ullah Jan, Azamatulla Khan
Reasons And Remedies For Time And Cost Overruns On Construction Projects - A Live Case Study
- Invited Talk by Engr. Rehan Ul Ambia Riaz, Consultant
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- Zeeshan Ullah, Muhammad Salman, Husnain Tariq But, Muhammad Saad, Mustansar Hussain
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Invited Talk by Engr. Arif Sattar, NED Alumni, Network, USA
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Potential drivers for adoption of green procurement in construction industry
- Ali Arsal, Aftab Hameed Memon, Nafees Ahmed Memon, Muhammad Akram Akhund, Ali Raza Khoso
Factors affecting selection of procurement method in public sector construction projects
- Izhar Hussain Bhutto, Nafees Ahmed Memon, Ali Raza Khoso, Muhammad Aslam Leghari, Shabir Hussain Khahro
Adaptation and perception of unmanned aerial vehicle in the construction industry
- Syed Abdullah Shah Hashmi, Shakeel Ahmed, Ehsanullah Kakar, Salah Uddin, Abdul Majeed

International Workshop on "Role of Youth & Young Professionals in Science Engineering, Technology & Innovation for Disaster Risk Reduction (DRR) on Sunday the 24th February, 2019 parallel of 10th International Civil Engineering Conference (ICEC-2019).

The following sessions were held including the opening session and the following were addressed in the opening session.
Engr. Prof. Dr. Sarosh Hashmat Lodi, Vice-Chancellor, NEDUET
Engr. Sohail Bashir, Chairman, IEP Karachi Centre
Engr. Prof. Dr. Abdul Jabbar Sangi, NEDUET
Mr. Raza Shah, Representative of UNESCO
Engr. Shoaib Ahmed, UNESCO-NEDUET
"Global Setting, Platforms and Networks on Youth and Young Professionals in DRR.
-Conducted by Ms. Irina Raflians, Indonesian Institute of Science (LIPI), Member Science and Technolgoy Advisory Group United Nations Strategy for Disaster Risk Reduction (STAG UXISDR)
"Youth and Young Professional in SETI for DRR: Challenges, Gaps, Needs, Expectations, and Initiatives"
-Conducted by Engr. Shoaib Ahmed, Assistant Professor, Department of Civil Engineering, NEDUET
"Lessons Sharing from U-INSPIRE Indonesia: Good Practices, management and organizational framework of U-INSPIRE Indonesia
-Conducted by Mr. Hilmar Arioaji,Vidographer, Vido Editor & Programmer as well as the Founder of Guerrilla Collective Indonesia. U-INSPIRE -Indonesia
"Role of Pakistani Youth in DRR"
-Conducted by Ms. Ghazala Naeem, Consultant. ICG/IOTWMS
"Recovering the Lost "Moment": How Timber-Laced Masonry May Hold the Secret to Stopping Pancake Collapse of Concrete Moment Frames (Part-2).
-Keynote Address by Mr. Randolph Langenbach, Conservationtech Consulting, Okland, California, USA on

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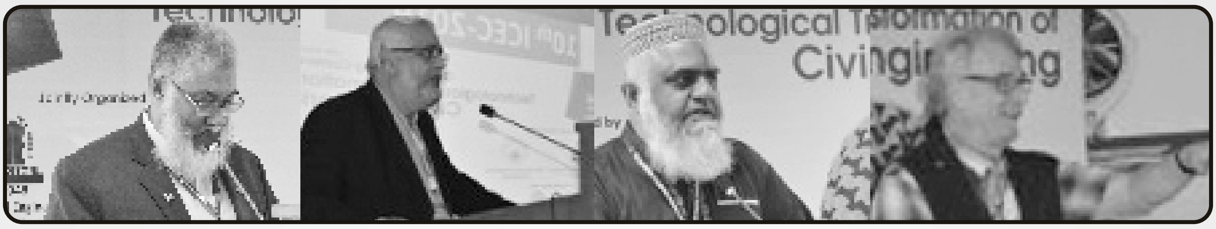

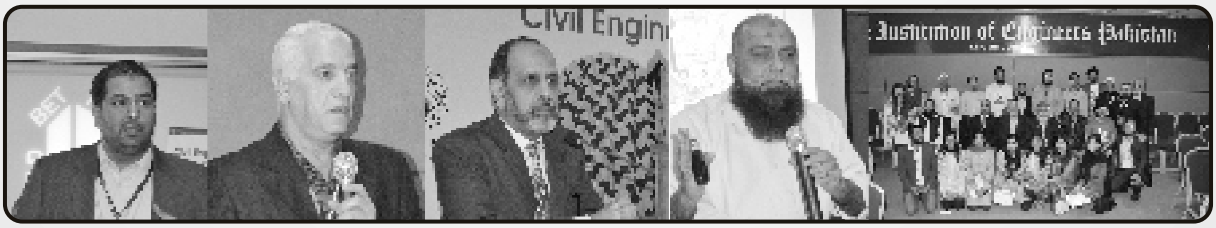

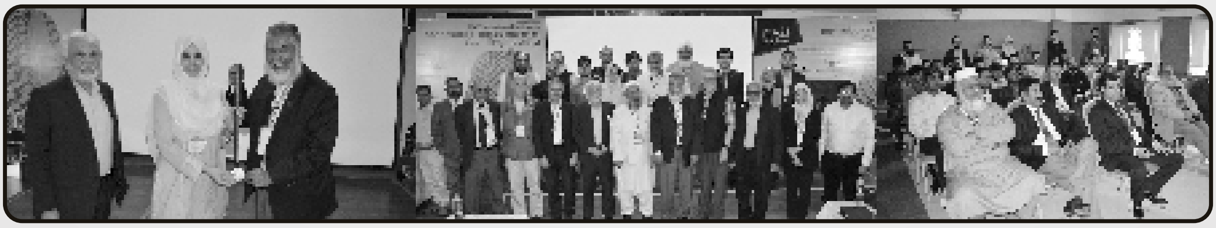

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