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TECH-CHRONICLE
AN INTERNATIONAL E-JOURNAL ON
EMERGING TRENDS IN SCIENCE, TECHNOLOGY AND MANAGEMENT

Sr. No	Name	Page
1	Mr. A. S. Rajas	1
2	Mr. Amarsinha Nitin Pawar	4
3	Ms. Ashwini A. Ketkar	8
4	Mr.Atul Sutar	13
5	K. C. Tayade	20
6	Manojkumar Shukla	26
7	Mohitsingh P. Katoch	29
8	Monika R. Jain	33
9	Durgesh.A.Khodankar	40
10	Kadambini Admane	44
11	Rahul M.Kachole	47
12	Rupali Bondre	50
13	Sneha S. Shende	53
14	Snehal S.Pawar	57
15	Sonali C. Patil	63
16	Sumit V. Bajare	68
17	Sweety R.Nagarkar	72
18	N. Y. Barve	75
19	Nitesh K.Jibhkate	78

20	Rajesh S. Jadhav	83
21	Sachin Rajendra Ingle	87
22	Soni Kumari	92
23	Prof. Swati R Dhurve	98
24	Swati M Sanap	105
25	Yogesh Survas	110

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Analytical Study of a High Rise Building Subjected to Wind Loading Using Different Bracing System

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Abstract: The focus of this study is on the wind analysis of high rise steel building with different bracing systems .This study will evaluate the efficiency of different bracing systems at different locations of steel building. In the recent past many high rise buildings or towers are being constructed in India. The impact of wind loads are to be considered for the analysis and design of steel towers or tall multi-storied building towers. We can minimize the storey drift and failure of these structures subjected to wind loading by using some retrofitting techniques such as different bracing systems. Tall structures are more flexible and susceptible to vibrations by wind induced forces. In the analysis and design of high-rise structures estimation of wind loads and the inter storey drifts are the two main criteria to be positively ascertained for the safe and comfortable living of the inhabitants .Inter storey drift can be controlled through suitable structural bracing system. Comparison of different bracing models with respect to deflection, Bending moment, Shear Forces, Axial forces in the different members will be done in this analytical study.

Keyword: High rise steel structure, wind loading, storey drift, different bracing systems

I. INTRODUCTION

High rise buildings have always been a dominant landmark in the townscape; at the same time has been preferred due to scarcity of land and to meet the increasing demand for space for residential and commercial purposes. These tall structures being slender light weight and with low structural damping undergo oscillations due to earthquake and wind loads. Wind is the term used for air in motion and is usually applied to the natural horizontal motion of the atmosphere. The horizontal motion of air, particularly the gradual retardation of wind speed and the high turbulence that occurs near the ground surface are of importance in building engineering. In urban areas, this zone of turbulence extends to a height of approximately one-quarter of a mile above ground, and is called the surface boundary layer. Above this height is called the gradient wind speed, and it is precisely in this boundary layer where most human activity is conducted. Therefore how wind effects are felt within this zone is of great concern.

Types of wind

Winds that are of interest in the design of buildings can be classified into three major types i.e. prevailing winds, seasonal winds, and local winds.

1. Prevailing winds

Surface air moving toward the low-pressure equatorial belts is called prevailing winds or trade winds. In the northern hemisphere, the northerly wind blowing towards the q\equator is deflected by the rotation of the earth to become north easterly and is known as the northeast trade wind. The corresponding wind in the southern hemisphere is called the southeast trade wind.

2. Seasonal winds

The air over the land is warmer in summer and colder in winter than the air adjacent to oceans during the same seasons. During summer, the continents become seats of low pressure, with wind blowing in from the colder oceans. In winter, the continents experience high pressure with winds directed towards the warmer oceans. These movements of air caused by variations in pressure difference are called seasonal winds. The monsoons of the China Sea and the Indian Ocean are an example.

3. Local winds

Local winds are those associated with the regional phenomena and include whirlwinds and thunderstorms. These are caused by daily changes in temperature and pressure, generating local effect in winds. The daily variations in temperature and pressure may occur over irregular terrain, causing valley and mountain breezes.

All three types of wind are of equal importance in design. However, for the purpose of evaluating wind loads, the characteristics of the prevailing and seasonal winds are analytically studied together, whereas those of local winds are studied separately

Characteristics of Wind

The flow of wind is complex because many flow situations

arise from the interaction of wind with structures. Following are the characteristics of wind

1. Variation of wind velocity with height.
2. Wind turbulence.
3. Statistical probability.
4. Vortex shedding phenomenon.
5. Dynamic nature of wind- structure interaction
6. From a structural engineer's point of view tall building or multi-storeyed building is one that, by virtue of its height, is affected by lateral forces to an extent that they play an important role in the structural design
7. Multi-storeyed buildings provide a large floor area in a relatively small area of land in urban centre

Bracing System:-

A bracing system can be defined as a structural system capable of resisting horizontal actions and limiting horizontal deformations. The main purpose of a bracing system is to provide the lateral stability of the entire structure. It has to resist, therefore, all lateral loading due to external forces, e.g. wind, imposed deformation, e.g. temperature, earthquake and the effects of imperfections on the simple bracing. For a non-sway frame, the bracing system must, in addition, be stiff enough so that second order effects need not be taken into account in the analysis.

Many types of bracing systems such as V, K or Chevron and X-type, Diagonal type bracings have been developed in order to comply with structural design requirements as well as architectural demands.

X-Braces as shown in figure (a) can increase the lateral stiffness of buildings and decrease the lateral displacements.

Diagonal or shear bracing as shown in figure (b) helps prevent buildings from "racking" or swaying.

Chevron-bracing as shown in figure (c) frames have been also developed in order to resist transverse dynamic loads.

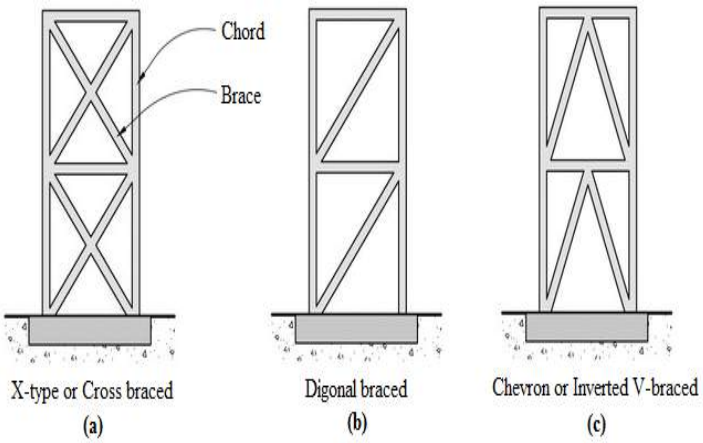

Figure 1: Different types of bracing system

II. PREVIOUS WORK

P. Suresh, B. Panduranga Rao, Kalyana Rama (2012), have analytically calculated the wind load using static and gust factor method for a sixteen storey high rise building and results were compared with respect to drift and it has concluded that in high rise buildings the stability can be achieved by suitably adding the dimensions of the corner columns with corner diagonal X-bracings. Provision of X-bracings reduces the amount of drift and bending moments in the structure. Provision of corner bracings can also be used as a retrofitting technique to strengthen the existing structure as X- bracings will act more like shear walls.

Daniel Christopher Berding (2006) studied literature and taken a comprehensive review which covers all pertinent aspects of wind drift in steel framed buildings. Next an analytical study of the variations in modeling parameters is performed to demonstrate how simple assumptions can affect the overall buildings stiffness and lateral displacements. This was investigated these sources of discrepancy through a thorough review of the literature, an analytical study of a typical 10 story office building, an analytical study on the sources of member deformations, and by developing a survey to assess the current state of the professional practice. In other words, this thesis was undertaken and written with the intention of suggesting and establishing a comprehensive, performance based approach to the wind drift design of steel framed buildings.

B. Dean Kumar and B.L.P. Swami, (2010), have calculated the wind loads using Gust Effectiveness Factor Method, which is more realistic particularly for computing the wind loads on flexible tall slender structures and tall building towers. In their work they have taken the frames of different heights of building are considered for analysis and study purpose and after analyzed the building frames It conclude that as the height of building frame increases, the energy content in the fluctuating component of wind also increases and the wind pressures computed by the gust effectiveness factor method are not only safer for design but also they are more rational and realistic.

Shahrzad Eghtesadi, Danesh Nourzadeh, Khosrow Bargi (2011), carried out the investigation and comparison with various types of bracing systems. For this reason, they have chosen four types of bracing systems include X-bracing, Diagonal bracing, Inverted chevron CBF and Inverted chevron EBF, in four different height levels, are modeled and analyzed in the STAAD pro software. After comparison of results. it stated that Application of the inverted chevron concentric bracing system may be proper and economical for the steel braced frames.

III. PROPOSED METHODOLOGY

For this study, a steel building of fifteen storey subjected to wind load in wind zone-III will be considered. A wind load will be calculated by Gust factor method using IS 875(PART 3):1987 as estimation of wind load by gust factor

method is more precise. Shear forces, bending moment, storey drifts and axial forces will be computed. There will be different bracing systems, e.g.- X-bracing, Diagonal bracing and Chevron Bracing system will be used and placed at different location such as At corner, At middle and Alternately. All the results obtained will be compared. In this regard STAAD Pro 2007 software package will be considered to perform analysis and Design.

IV. EXPECTED RESULTS

From most of the research papers referred it can be seen that 'X-bracing' is more efficient in reducing lateral displacement of structure and will have minimum possible bending moments in comparison to other types of bracing systems. When X-brace is used it gives the highest capacity but comparatively less ductility. The chevron type of brace gives moderate performance during an earthquake, since the capacity and ductility both are achieved. The location of brace member has significant effect on the seismic response of the braced frame structures. The central locations of brace member are favorable as they are effective in reducing actions induced in frame with less horizontal deflection and drift. Comparison of static effect and wind effect on the structure in all the parameters can be found based on the above study. From this study, most efficient type and location of bracing system can be found out for the typical structure considered.

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Studying the Present Contract Model and Causes of Delay in the Model

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***Abstract*-Public works are done at the cost of public funds and it is done for the public itself but due to delay in contract execution a lot of money is drained from the public account which is a great loss. There are various reasons for projects getting delayed in public sector and is the biggest problems faced by the construction industry today. One of the important reasons for delay in project is the procurement procedure followed. Studies have been made on the reasons in construction delay from the client, consultant and contractor point of view using statistical formulae but an empirical study needs to be done on the whole procurement procedure followed in the public sector. The general flow of present procurement model followed is project idea, its preliminary report, detailed project report, technical sanction, conformation of funding, preparing of tender document, bidding process and selection of contractor. This process though important is mainly time consuming. If this process is redesigned than issues related to delay can be reduced. The time taken for sanctioning of a project is too lengthy because the project report has to be passed by every concerned department. The estimates at the sanctioning stage and the actual cost of work at the time of execution are different rather are increased substantially which leads to cost escalation. The basic principle of fair competition i.e. to award the contract to the lowest bidder needs to be changed with alternative method. Issues related to cash flow, billing, dispute resolution board needs to be addressed. The aim of the paper is to study the present contracting model. Based on the study giving recommendations to the present contracting model and proposing a new model. Resulting in reducing the procedure time, selecting appropriate contractor, avoiding disputes and saving the public money.**

Keywords- Contract Delay; Procurement Procedure; Time Overrun; Cost Escalation

I. INTRODUCTION

Time is the essence of contract. The inability of completing public sector civil engineering projects in time and within the budget is one of the chronic issue now a days. New utility services comes with new service need and increasing standard of living. Public works like roads, railways, bridges, ports, etc. are done at the cost of public funds and for public itself but due to delay in contract execution a lot money is drained from the public account which is a great loss. With delay in projects come the unhealthy conditions as well which has to be faced by the public. Hence it is necessary to reduce the delays in public sector projects and make appropriate use of public funds.

Public sector civil engineering projects are seen not getting completed in time and within the allotted budget. One of the example is, Bandra-Worli Sea Link which was planned for Rs 300 crore and with completion date of 4 years but actually cost went up to Rs 1600 crore along with delay of 5 years. Such example are common now a days, there is need to study the pre contractural procedure followed so as to reduce the delay in administrative procedure and cost escalation issues. The study of the is on the procurement procedure followed in the public sector for civil engineering work. The aim is to prepare existing contract model and find the causes of delays in the model

II. OBJECTIVE

The objective is to study the procurement procedure followed by Municipal councils in Pune region for public civil engineering works. Preparing a model of the existing system and find the causes of delay in it.

III. NEED

There are many research work available on construction delay focusing on execution delays alone. There is need to study the procurement method followed in the public sector civil engineering works. There are many issues related to time overrun and cost escalation in the public project, they can be reduced by studying and analyzing the procurement method followed for these works.

IV. LITERATURE REVIEW:

Rarer are the studies based on completed projects. As a result, the extents as well as the causes behind delays and cost overruns have remained under-researched (1). The organizational failures are significant causes of delays and cost overruns in public sector projects.

Claims occur due to disputes between the parties where the method of disruption claims and delay will be fixed. This will avoide the conflicts later, since the negotiation will be binding on the parties in contract.

The fair competition view in the procurement system has lead to the selection of lowest bidder in awarding of contract. The bidder may not be capable of carrying out the

work efficiently in future leading to time and cost overruns. The lowest bidder win method must be changed with an alternative method.

The delays can be reduced with less pain if areas like detailed engineering is performed before detailed construction estimate is prepared, developing good execution drawing, more engagement of consultancy for better design and reducing time at execution date (4).

The five most important causes of delays in construction projects were found to be: change of work scope; delayed payments; poor monitoring and control; high cost of capital; political instability/insecurity. Recommendations were made for improved project management; change from the traditional contract type to the design-build type; and improved cash flow on the part of the client so as to reduce payment delays (6)

V. RESEARCH METHODOLOGY

Studying the literature related to the topic. Collecting the data like tender document, standard form of contract, discussion with the experts in the contract field. Preparing a model based on the information gathered from these sources. Identifying the causes of delay in the model.

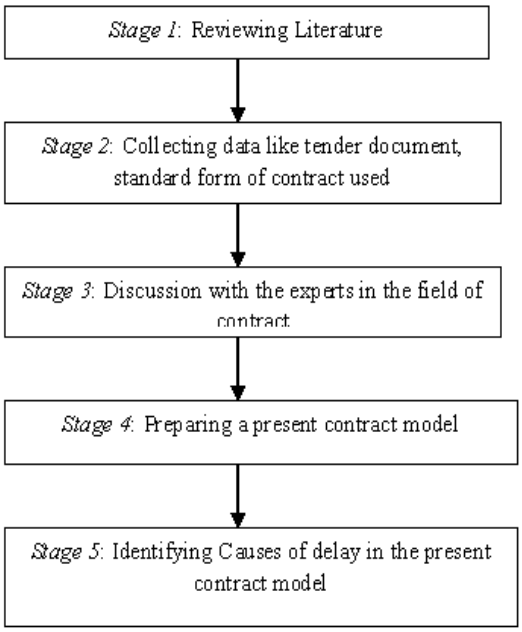


Fig. 1. Flow Chart of Methodology

VI. FINDINGS:

Initially when the need arises of providing new service, the respective department proposes the preliminary project report in front of the council. The estimates prepared at this stage are tentative estimates. Once it is kept in front of the council a resolution is passed regarding the proposed work and first reading in the council takes place. If the proposed work is accepted in the first reading than it is taken to second reading where the work gets conformed from the council. If the proposed work is not accepted in the first reading than it goes out of the process. Once the conformation has been got from the council it is send for preparing a detailed project report. Once the detailed project report is prepared it is send to town planning department for approval of plan and Maharashtra Jivan Pradhikaran or Public Works Department for approval of technical sanction and estimates. These two activities go hand in hand. After passing through these departments than the town planning department checks the provision of work, type of work and then selects a government scheme suitable to the work. Time consumed only for sanctioning as per the scheme, is around 3 months. Once the funds are arranged the work proceeds for tendering process. Time consumed for clearing stage 1 is around 9 to 10 months.

Tenders are called upon and bidders are invited for bidding. Once the bidders are short listed their offer has to be selected within the validity period. Validity of bid is the certification from a bidder of the period of time their bid can be considered valid. After this period, the bidder is at liberty to change their bid price if the contract is not signed by the last date of the bid validity period. Bid validity period is the period within which a bidder offer is considered legally binding. After this period, the bidder is at liberty to change their bid if the contract has not been signed. If validity period is crossed without any action than the tenders are to be called again. If the bidder's offer is accepted than the work proceeds to selection of contractor. While selecting the contractor his bid capacity is checked and then only work is given to him. Once the contractor is selected work order is issued to him. Time consumed for clearing stage 2 is around 2 to 3 months.

Once the work order has been got by the contractor he starts to execute the work. The payment stages are fixed in the contract. Stage wise payment is done to the contractor. Whenever any disputes arises while execution of the work, these disputes are taken to the dispute resolution board. The decision of the dispute resolution board is binding on the parties in contract. The contractor gets benefit if the price level rises or falls in the period of execution of the work but this is not applicable to small works where the completion period is less than eighteen months. The contractor gives the monthly progress report of the work to the authority. The bills along with the material testing report are submitted as well.

The model has been divided into three stages for better understanding:

- Stage 1: Administrative Approvals and Fund Raising (Time consumed is around 8-10 months)
- Stage 2: Tendering Process (Time consumed is around 2-3 months)
- Stage 3: Execution of the Work

Following flow charts represents the present model of contract followed in general for public civil engineering works:

Fig. 3. Flow chart of stage 2 of the contract

Stage 1: Administrative Approvals and Fund Raising (Time consumed is around 8-10 months)

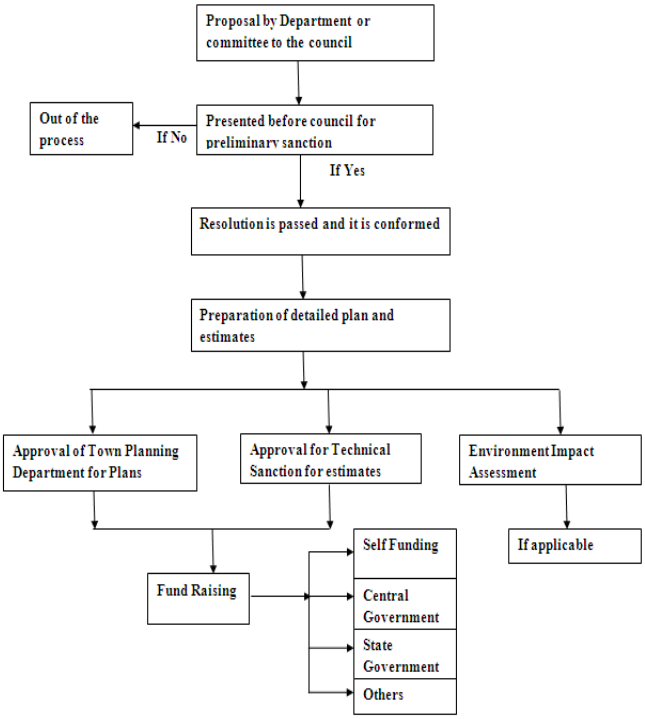
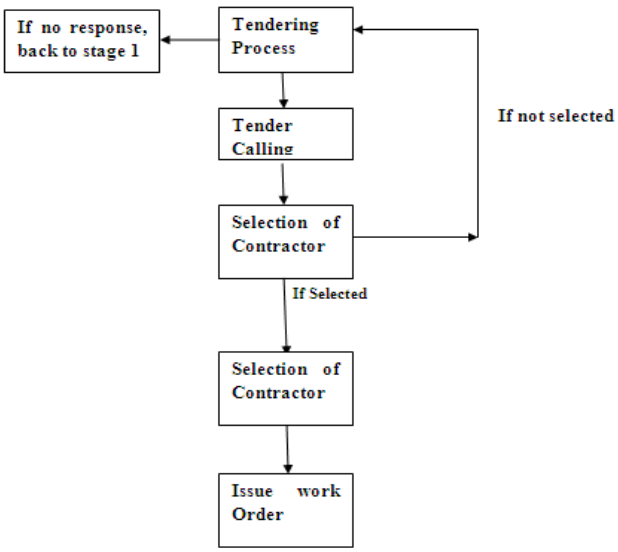


Fig. 2. Flow chart of Stage 1 of the contract

Stage 2: Tendering Process (Time consumed is around 2-3 months)



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Stage 3: Execution of the Work

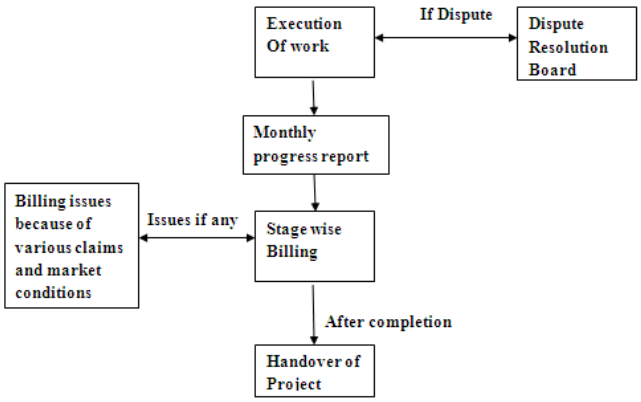

Fig. 4. Flow chatr of Stage 3 of the contract

VII. CONCLUSION:

The delays can be categorized into three categories as system delay, contractors delay and delay from social elements.

A. System delay

The time consumed for administrative procedure is around 6-7 months. this includes first proposal of the work from department to sanction it from town planning and MJP or PWD department. Sanctioning of funds from various available schemes takes around 3 months. So here itself before tenders are called time consumed is around 9-10 months. Hence the detailed estimates prepared while sanctioning may differ at the initial stage before calling of tenders itself. District Schedule Rates are released in the month of March-April and if tenders are called around this period there is no response to the tender process and then the process has to be started again from estimate preparation stage and again sanctioning the work. Many a times the contracts are not signed within the validity period. If they are not signed within the validity period than bidder is free to change his rates and tenders are to be called again leading to cost escalation from the initial estimated cost. The tendering process is based on fair competition, hence lowest bidders are selected. Many a time's lowest bidder is not able to carry out the work and work stops adding to delay. In this situation a new contractor may be appointed increasing time and cost of the project. From tender floating to issuing of work order around 2 to 3 months time is consumed. The bid capacity of the bidder is checked after the bidder has submitted for the work. The contractor does not get his bill as per the stages in the contract this leads to stopping of the work

from contractor adding to delay and time overrun. He does not start the work unless his bill is paid by the public authority. For billing purpose still the hand written measurement books are used which is very tedious to fill up and lot of time is consumed which delays the billing procedure. Dispute resolution board consist of the members form the working committee of the project and contractor is bind to listen to this committee in case of any dispute arises.

B. Contractor's delay

The tendering work is based the principle of fair competition. Generally the lowest bidder gets the work. These criteria many a times make lots of problem during the execution of work. Availability of the material in the market at the expected cost, is not assured, hence many a times the work cannot progress further or contractor demands price escalation. The contractor may not have good credit than he is likely to suffer due to rate fluctuation from the market. Ego problems of the contractor i.e. in the competition the contractor rates the bid at low rates which cannot be expected. This may be because of (i) his need to get work; (ii) not allowing a new competitor to get into the competition;(iii) attitude of the contractor towards the work. Cash flow issues of the contractor. Contractors take loan against the work order, if the billing is not done regularly than contractor may go towards bankrupting. Banks make their attachments to the bills of the contractor leaving contractor without money to carry out the further work. Even though the work order is received, the contractor does not start the work, this issue must be addressed. Since the time consumed from floating of tenders to issuing of work order is around 2-3 months, where contractor is at loss because of changing market condition.

C. Delay from Social Elements

Social elements like right to information from activist, disputes arising from land acquisition , court matters, pressure group activities and act of god affects the progress of the work adding to delays.

VIII. Future Scope of the work

Present work has identified the procurement model of the public sector civil engineering work followed. Further work can be done by proposing new model which will help in reducing time overrun and cost escalation issues.

IX. Acknowledgement

I express my profound gratitude toward respected Mr. Jayant Kikale, Proprietor of Kikale Associates, Baramati, Mr. Milind Kapile, Executive Engineer, Electrical Department PCMC and Mr. Sanjay Bhosle, Deputy Engineer, Civil Department PCMC for their constant encouragement and valuable guidance during completion of my work. They have been a strong and moral support to me throughout this work.

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Prediction of severity of environmental impacts related to construction process on residential building – A Review

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Abstract— The construction is considered as main source of an environmental pollution. The demand of the construction is increasing day by day in the world. The main aim of this study is to evaluate the most common environmental impacts on residential building during construction. The model study will help the organization to identify the most commonly environmental impacts according to their impact level, frequency and consequences of occurrence. The study will help the organizations and project manager or construction manager to increase the knowledge of people which are involved in construction through training and awareness program. The aim of this paper is study of implementation of EMS (Energy Management System) in construction companies and to improve the organizational environmental performance.

Different methodologies are able to compare the overall impact of various construction projects and rank the significance of various environmental impacts on each project. The environmental impact at particular site is identified primarily at construction stage and their significant impacts are highlighted. Thus, it is possible to implement measures for mitigating adverse impacts, which can then be implemented during construction activities. The paper will serves as an assessment tool for the construction project to measure the inputs of their construction activities.

1. INTRODUCTION

Construction is considered as one of the main sources of environmental pollution in India. With inadequate environmental planning, human activities have resulted in the disruption of social and communal harmony, the loss of human livelihood and life, the introduction of new diseases, and the destruction of renewable resources. Therefore it is necessary to evaluate the most common environmental impacts due to construction processes.

EIA is a planning tool that is now generally accepted as an integral component of sound decision-making. The objective of EIA is to foresee and address potential environmental problems/concerns at an early stage of project planning and design. EIA systematically examines both beneficial and adverse consequences of the project and ensures that these effects are taken into account during project design.

It helps to identify possible environmental effects of the proposed project, proposes measures to mitigate adverse effects and predicts whether there will be significant adverse environmental effects, even after the mitigation is implemented. Environmental Impact Assessment can thus be defined as "the systematic process of identifying future consequences of a current or proposed action." EIA is both an art and a science. Management aspect in EIA is an art, whereas the technical analysis is based on the scientific principles.

1.1 EIA in Industry

The purpose of Environmental Impact Assessment (EIA) is to identify and evaluate the potential impacts (beneficial and adverse) of development and projects on the environmental system. It is a useful aid for decision making based on understanding of the environment implications including social, cultural and aesthetic concerns which could be integrated with the analysis of the project costs and benefits. This exercise should be undertaken early enough in the planning stage of projects for selection of environmentally compatible sites, process technologies and such other environmental safeguards.

While all industrial projects may have some environmental impacts all of them may not be significant enough to warrant elaborate assessment procedures. The need for such exercises will have to be decided after initial evaluation of the possible implications of a particular project and its location. The projects which could be the candidates for detailed Environment Impact Assessment include the following:-

- Those which can significantly alter the landscape, land use pattern and lead to concentration of working and service population;

- Those which need upstream development activity like assured mineral and forest products supply or downstream industrial process development;

- Those involving manufacture, handling and use of hazardous materials;

- Those which are sited near ecologically sensitive areas, urban centres, hill resorts, places of scientific and religious importance.

1.2 EIA in India

The Ministry of Environment and Forests (MoEF) of India has been in a great effort in Environmental Impact Assessment in India. The main laws in action are the Water Act(1974), the Indian Wildlife (Protection) Act (1972), the Air (Prevention and Control of Pollution) Act (1981) and the Environment (Protection) Act (1986),Biological Diversity Act(2002). The responsible body for this is the Central Pollution Control Board. Environmental Impact Assessment (EIA) studies need a significant amount of primary and secondary environmental data. Primary data are those collected in the field to define the status of the environment (like air quality data, water quality data etc.). Secondary data are those collected over the years that can be used to understand the existing environmental scenario of the study area. The environmental impact assessment (EIA) studies are conducted over a short period of time and therefore the understanding of the environmental trends, based on a few months of primary data, has limitations. Ideally, the primary data must be considered along with the secondary data for complete understanding of the existing environmental status of the area. In many EIA studies, the secondary data needs could be as high as 80% of the total data requirement. EIC is the repository of one stop secondary data source for environmental impact assessment in India.

2. EIA PROCESS

The course of action in an EIA depends on the prerequisites of the nation or patron, though, as a rule EIA processes have a regular constitution and the implementation of these main steps is an essential criterion of first-rate preparation. The environment impact assessment entails eight steps wherein each step is equally significant in influencing the overall performance of the project. Characteristically, the EIA progression commences with screening to certify that the time and reserves are concentrated at the proposals which matter environmentally and concludes with some kind of follow up on the execution of the verdicts and actions taken as an outcome of an EIA report.

The eight steps implicated in EIA process are:

1. **Screening:** This first stage of EIA establishes whether the anticipated project, necessitates an EIA and if it does, then the intensity of assessment required.
2. **Scoping:** This step recognizes the key issues and impacts which further needs to be investigated and also delineates the periphery and time limit of the study.
3. **Impact Analysis:** The third stage of EIA identifies and envisages the likely ecological and social impact of the anticipated project and weighs upon the implication.
4. **Mitigation:** Mitigation advocates the actions to lessen and steer clear of the possible adverse environmental upshots of development activities.
5. **Reporting:** The end result of EIA is accounted in a form of a report to the decision-making body and other concerned parties.
6. **Review of EIA:** It reviews the sufficiency and efficacy of the EIA report and makes available the information essential for decision-making.
7. **Decision-Making:** It decides whether the project is cast off, accepted or needs additional alterations.
8. **Post Monitoring:** This final step of EIA comes into play once the project is bespoke. It tries to make certain that the impacts of the project does not go beyond the officially permitted standards and execution of the mitigation measures are done in the manner as stated in the EIA report.

3. REVIEW OF PREVIOUS PAPERS

Peyman Karami et al. [1] presents the result of life cycle analysis (LCA) study that compares the environmental impacts of three hypothetical buildings, a standard residential building, a regular well insulated building and a building insulated with VIPs. The results of his research illustrated the environmental impact of production stage of the various building materials and components. He concludes that in general increased thermal insulation will help to reduce the energy use and the environmental impact of buildings. He observed that VIPs have greater environmental impact than conventional insulation in all categories except ODP (Ozone Depletion Potential). He concluded that VIP core material with a great contribution of about 90% of the total environmental impact have a significant potential for reducing the environmental impacts of a manufactured VIP. He suggested that further investigations require for minimizing the measurable environmental impacts that acquired in this LCA study for well insulated building with VIPs.

Jaehun Sim et al. [2] assess seven types of air emissions and then demonstrate the application of the proposed methodology to a recently constructed apartment building in Busan, South Korea. He estimates the quantity of seven of air pollutants – Carbon Dioxide (CO2), Carbon Monoxide (CO), Methane (CH4), Nitrogen Oxides (NOx), Sulphur Dioxide (SO2), non Methane volatile organic compounds (NMVOC) and Nitrous Oxide (NO2) produced during the life cycle of an apartment building in South Korea. He concludes that steel is the largest source of CH4 emissions and NO2 emissions while the concrete is the largest source of CO2 emissions. He suggested to further expands this study to estimate total air emissions produced from the nations entire building sector.

Ali El Hanandeh et al. [3] studied the six most popular construction configurations for a single family house in Jordan are assessed using "cradle to grave" life cycle methodology. The alternatives included single hollow concrete block (economics), double layer that concrete block with insulation layer (insulated economics) – typical limestone cladding, insulated limestone wall and multi-layer with limestone cladding and insulation (luxury). He concluded that under low energy requirements for HVAC, the economics house with walls constructed from a single layer of hollow concrete blocks offers the best environmental performance from a life cycle perspective. He also elaborates that wall construction with insulation reduces energy use during the operation phase. It has higher environmental impacts over the lifespan of buildings when material production, construction, demolition and disposal phases are considered. He also suggested that water depletion and energy resources impacts be used as proxy indications of overall environmental performance of the single family house in Jordan.

Jui Sheng Chou et al. [4] developed a Carbon Dioxide emissions evaluations systems and an environmental cost calculations method. The simulations considered the consumptions of the fossil fuels, electricity and water. He was proposed a systematic approach for optimizing the balance of Carbon Dioxide emissions and environmental cost during building life cycle. His study presented a simplified metric for converting CO2 emissions into environmental costs. He proposed that his method can facilitate engineers and architects in evaluating primary environmental risks for building life cycles and selecting adequate construction methods.

Alba Fuertes et al. [5] aim is to improve the understanding of construction related environmental impacts by identifying onsite casual factors and associate immediate circumstances during construction processes for residential building projects. He developed a construction related environmental impact casual model consisting of a process oriented casual network of thirty nine environmental impacts, forty five casual factors and over two hundred casual relationships. His model will help to compliment the environmental managerial decisions, mostly related to the identification and assessment of the environmental aspects related to construction processes. He suggested that there are some opportunities for improvement to achieve better environmental performance levels for construction projects and reduce impacts onsite.

Ya Hong Dong et al. [6] developed a Life Cycle Assessment model (LCA) viz. the Environmental Model of Construction (EMoC) to help decision maker's asses the environmental performance of building construction projects in Hong Kong from cradle to end of construction. His model provides comprehensive analysis of 18 environmental impact categories at midpoint and end point levels. He concludes that the material is a major contributor to environmental impacts of upstream stages of public housing construction. He suggested the scope of EMoC is currently limited to upstream stages. It can be expanded to encompass the downstream process of operation, maintenance and demolition in future so that more accurate assessment on the environmental performance of a building project can be achieved.

Helena Dahbo et al. [7] was assessed the performance of the common finish C&DW management system against this target, thus identifying the environmental and economic impacts of the system and effects brought about by changes in the waste composition. He was applied combination of different methodologies and evaluates the performance of the C&DW management system. He was employed (MFA) material flow analysis to assess material environmental life cycle costing (ELCC) for measuring the costs and life cycle assessment (LCA) to evaluate climate change impacts. He provides new information on the factors affecting the results of environmental and economic performance of C&DW management system.

Hamidul Islam et al. [8] describe an optimization approach of balancing life cycle costs and environmental impacts for typical Australian houses. Two sets of objective functions viz. life cycle costs and environmental impacts were evaluated for each design using life cycle costing and life cycle assessment approach respectively. He concluded that a house designed with these optimal wall, floor and roofing had a much higher rating (by 1.5 stars) lower environmental impacts (by 10-20%) for the same life cycle costs. He suggested the approach would be useful to identify building elements that could achieve even higher star ratings.

Mohamed Marzouk et al. [9] evaluate the impacts of two alternatives for the management of CDW, recycling and disposing. They developed a dynamic model by using STELLA software. The research findings show that recycling CDW leads to significant reductions in emissions, energy use, and global warming potential (GWP) and conserves landfill space when compared to disposal of wastes in landfills. He suggested that it is necessary to recycle construction and demolition wastes. He proves that the cost incurred to reduce the danger to the environment and human health due to uncollected waste and waste landfill is extremely high.

Marta Gangolells et al. [10] were identified 42 onsite environmental impacts of the municipal engineering works by means of process oriented approach. Then 46 indicators and their corresponding significant limits were determined on the basis of statistical analysis of 25 new build and remodeling municipal engineering projects. He suggested that preventive actions can be planned and implemented during onsite activities. Further research should also examine the socio-economic impacts related to municipal engineering projects and investigated the possibility of including them within the developed methodology.

Md. Shahrior Alam et al. [11] show environmental impacts of a building from its construction phase to the end of its life. i.e. demolition. He estimates the CO2 emission from various building plans for its entire life, which help us in guiding the reduction of CO2 emission from a building. His analysis will encourage us to analysis mode the environmental impact of a building and will show the importance of reduction the emission of CO2. He also concludes that bricks and natural gas produce most of CO2.

Tao Huang et al. [12] estimated the material demand and environmental impact from buildings in China from 1950 to 2050 based on MFA. He also estimated the effect of prolonging the lifetime of buildings construction and material recycling on reducing raw material consumption and CO2 emissions. He suggested that a large number of solid used and recycled materials from building construction should be used in transportation infrastructure construction. In this study, it is only limited to recycling of scrap steel and waste aggregates in the building system.

Francois Nemry et al. [13] developed a typology of building representatives of the building stock of EV-15. The building types were selected and characterized in terms of their geographical distribution, size, age, design, material composition and thermal insulation. For existing buildings the three major additional façade insulation, additional roof insulation and new sealing to reduce ventilation yield a significant environmental improvement potential which for a majority of building represent at least 20% compared to the base case.

Varinder Saini et al. [14] conclude that the Geo-Environmental parameters such as air, water, soil, agricultural lands, vegetation, landforms are affected by directly coal mining activities. He attempted to review the EIA studies done in various coal fields in India. He demonstrated that conjunctive application and analysis of field based measurements, expert opinions and remote sensing data may be useful for carrying out environmental impact studies in mining areas. Various remedial measures for reducing the pollution, restoring coal mining areas using different methods have been suggested by several workers but discussing them in detail here would have been out of the scope of this paper.

Malindu et al. [15] established a process based quantitative method to estimate emissions due to materials, transportation and equipment usage. Results obtained an average GHG emission of 67%, 19% & 14% from materials, equipment and transportation respectively. His observation signifies the relative higher % of emission distribution of equipment and transportation in foundation construction compared to that in total building construction. The results obtained in case studies will be utilized to carry out further research on emission at total building construction stage. It is also intended to further investigate emission reduction options at total building construction stage. One major assumption is that the study only considers emissions related to construction work and neglects indirect emissions due to lighting, heating and cooling.

Bianca et al. [16] analyzes the trend in the adaption and implementation of the environmental policies and assessment procedures by the major international institutions since the introduction of environmental assessment concept in the legislation of several countries. In the intention of the authors, therefore this review paper is meant as on opportunity for the impact assessment community to reason on the foundation of ESIA and start. They also intended to raise attention on an assessment process that responds to a widely shared need for the evaluation of social issue to come into practice.

Alison Rothwell et al. [17] were identified integrated land use options for PU regions in which housing and feeding an expanding population can both be achieved with lower environmental impacts that is the norm for Greenfield development. He has given examples of alternatives food and housing land-use scenarios in the Pu context. Furthermore, he suggested that the housing and food production typologies can be used to determine the attributes of the system that will result in a desired reduction in their environmental impact, such as for climate mitigation purposes.

Xinying et al. [18] concluded that to fully understand the EI of PRB construction in China, this study chose two typical residential buildings that were constructed with the prefabrication technology and traditional cast-insitu technology for a comparative study. He concludes that prefabrication technology demonstrated a certain degree of advantages in EI compared with the cast-insitu technology. His research could provide useful information for the government to make targeted measures to promote prefabrication technology in the Chinese residential industry.

4. APPLICABILITY OF EIA TO SMALL AND MEDIUM PROJECTS

In broad, EIA process calls for assortment of resources which include personnel resources, funding, time to perform entire task, and can be done only for large sized projects which are attaining retorts from community. However there exist a few projects which may possibly have an effect on the environment unremittingly owing to their routine activities. These projects have an increased need of natural resources. Furthermore they devour energy, generate solid waste and impinge on the close by land sources enduringly. These small and medium level projects need attention considering the fact that development of a country depends on the small projects whose ratio is always more and are instigating on a regular basis.

5. CONCLUSION

- In view of the fact that development is an ever growing process, its impact on the environment is also ever increasing, leading to rapid deterioration in environmental conditions. As such Environmental Assessment provides a rational approach to sustainable development.
- Extensively developed rapid assessment techniques often avoid carrying out of detailed studies which need more resources in terms of time and money. This exercise is an attempt in developing an approach to Environmental Assessment technique, primarily for residential buildings.

- Environmental Impact Assessment should not be a barrier to growth and will only apply to a small proportion of projects considered within the town.
- The aim of Environmental Impact Assessment is also to ensure that the public are given early and effective opportunities to participate in the decision making procedures.

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Prototype construction project management information system: Performance & Evaluations

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Abstract: The Indian construction industry is known to be inefficient and highly resistant to change. Even with a changing market and increasing competition, there are no obvious signs of commensurate changes in methods and approach. Project management, on the other hand, seems to offer what is needed in terms of tools and techniques to raise industry standards. Currently of the project management based systems are being implemented by using web based Information system. This paper represents the efforts made on project management by Internet. Information technology is routinely used in construction industry as tool to reduce efforts, solve problem with improved coordination, Primavera and Microsoft projects (MSP) software are used here for construction project management Information system.

Keywords: *project management Information system, Information technology, construction management, web based system, Construction industry.*

1.1 GENERAL

Construction works are carried out in the form of project. Projects are becoming progressively larger and more complex in terms of physical size and cost. In the modern world, the execution of a project requires the management of scarce resources; manpower, material, money, and machines to be managed throughout the life of the project – from conception to completion. Most of the time the Construction Industry has been criticized for its under-performance due to lack of performance measurements, project monitoring productivity, cost effectiveness, safety and sustainability. The construction industry business process is project based with multiple parties involved, including owners, general contractors, architects, engineers, sub-contractors and material suppliers. It needs to communicate on a large scale with these parties. In contrast to other industries this industry it is not exposed to new technologies and methods every day exception to some new materials. This has resulted in the state that, existing technologies and methods have percolated to the root level and the industry does not find the need of high end professionals for day to day activities.

The construction industry is characterized by many players of multiple disciplines who are brought together at various stages throughout a single project. The result is a reliance on a large body of information produced by many sources at many levels of abstraction and detail, which contributes to the fragmentation of the industry. This fragmentation, in turn, contributes to the poor record of overall productivity improvement in the industry. The construction industry has been faced with the problems of meeting project schedule, budget, safety, quality and specifications set by the owner and architect/engineer. The proper utilization of internal and external resources is essential if construction companies are to make the best business decisions, maximize business goals and survive in the competitive environment. Although the construction industry is one of the most highly fragmented, inefficient, and geographically dispersed industries.

ERP system could be used in the construction industry for the following purposes:

1.To improve responsiveness in relation to customers;

2. To strengthen supply chain partnerships;

3. To enhance organizational flexibility;

4. To improve decision making capabilities;

5. To reduce project completion time and cost.

These information systems are designed to integrate and partially automate many of the company's business processes such as human resources, financial management, manufacturing, procurement, construction, operation and maintenance. It has been found from the literature review that out of the total cost, system-based costs including software cost, averages 40% of the total cost, the remaining 60% of the cost goes to training and professional services. ERP software cost averaged a mere 15% of the total cost of the system implementation.

1.2 INDIAN CONSTRUCTION INDUSTRY

The construction industry was accorded Industrial Concern Status under the Industrial Development Bank of India (IDBI). Now, the construction industry is the second largest industry, next to agriculture. With increasing thrust on developing infrastructure and attractive concessions appeasing private partnership in infrastructure projects, the Indian construction is already booming and is poised to see a bigger growth in future.

Construction is an integral part of infrastructure such as houses, offices, townships, urban infrastructure, highways, ports, railways, airports, power, irrigation, industrial project & so on. The importance of construction can be gauged from the simple fact that cost of construction of certain infrastructure projects may be as high 60 percent to 80 percent of the project cost. Construction is recognized as the basic input for socio-economic development. It generates substantial employment. .

2.LITERATURE REVIEW

2.1 INTRODUCTION

Project management attempts to achieve project mission objectives within specified constraints. It needs information to make decisions. Information plays an important role in binding the building blocks of modern multidivisions, multiplication and multinational organizations.

The literature survey carried out is divided into a number of sections

1) The basic concepts of management information system
2) Management Information System Framework
3) Specification For Developing Information system
4) System Acquisition
5) Problems in Information System Management.

2.2 MANGEMENTS INFORMATION SYSTEM CONCEPT

In the following paragraph, MIS functions, its components, the structure of MIS, ways to improve management information system in an organization are discussed.

2.2.1 MIS FUNCTIONS

Project Management Information System is an integrated user-machine system that provides information to support operations, management and decision making functions relating to planning and control of project objectives. It main functions are To set standards against which to measure and compare progress and costs. These standards include project time schedules, project control budgets, material schedules, labor schedules, productivity standards, and quality control specification and construction drawings.

1. To organize efficient means of measuring, collecting, verifying and quantifying data reflecting performance with respect to time, cost, resources and quality.

2. To manage means of converting data from operations into information.

3. To report the correct and necessary information in a form, which can best be interpreted by management, and at a level of detail, most appropriate for the individual managers or supervisors who will be using it.

4. To provide management 'exception reports' to highlight critical factors.

2.2.2 MIS COMPONETS

Project Management Information System comprises hardware, software, database, procedures, operators and documents.

Hardware The term hardware covers all the electronic and electro- mechanical equipment used in computerized data processing systems. This equipment consists of the Central Processing Unit **(CPU)** and its peripherals. Peripherals are the externally connected devices of the computer such as input devices, storage disks and output devices.

2.3 MIS FRAMEWORK

The information system supporting project management can be broadly categorized into five subsystems. These are

- Data processing system for operation and managerial levels.
- Decision support for managerial and top level
- Office information system for office and general purpose applications
- Artificial intelligence based systems for top level
- Communication system for co-ordination at all levels.
- The above classification is not rigid and a system may be composed of any one or a combination of the above five system.

2.4.1 SOFTWARE DEVELOPEMNT LIFE CYCLE (SDLC) FOR CONSTRUCTION SYSTEMS

The Software Development Life Cycle is the entire process of formal, logical steps taken to develop a software product. In other words, it is a framework for understanding and developing Information Systems and software successfully.

Various phases of software development life cycle

2.4.2 Recognition of Need/ Requirement Analysis

Understand the problem before solving it

Change in already existing system

This leads to initial preliminary investigation.

System Analysis produces statement of objectives of the candidate system

Not all proposed systems are desirable and sent for review.

2.4.3 Feasibility study

It is the outcome of the preliminary investigation and in this phase it is determined that the system is feasible or not System Analysis and his team members with the help of the user carry out feasibility study. The outcome of feasibility study is a formal proposal detailing the nature and scope of the proposed system containing the statement, problem, summary of findings and recommendations.

After the management reviews the proposal, it becomes a formal agreement, which then proceeds to the next step of Software design life cycle.

There are three aspects of Feasibility Study:

2.4.4 Analysis

It is the detailed understanding of all-important aspects of the system and their relationship within and outside the system.

Scope of proposed system is decided using data gathering tools such as interviews, onsite Observations, Questionnaire, etc.

The next step is to decide how to solve the problem.

Till this stage logical design of stem finishes and system enters the physical design stage.

2.4.5 Design

This phase produces the details of how a system will meet the requirements.

Design stage describes the data to be accepted, calculated, data storage method, table design with validation and security processes.

Data designers are responsible for providing programmers with complete and clearly stated **System Requirement Specifications (SRS).**

Reports formats are decided at this stage with the interaction with the user.

2.4.6 Implementations

During this phase the system actually take physical shape. Implementation stage consists of writing programs, testing and documenting programs, conversion of old data to new system, training the users, installation of hard ware and software if necessary and documenting entire system.

Evaluation of system as whether system fulfills original needs is also done.

2.4.7 Post- Implementation and Maintenance

Like any system there is an aging process that requires periodic maintenance of Hardware and Software.

During maintenance, programmers may find some exceptional conditions which have to be rectified or modified.

Developing system that requires as less maintenance as possible should be the primary goal of the System Development.

2.5 SYSTEM ANALYST

System Analyst designs Information System, which meet the organizational objective, promote integration of activities, and facilitate control and which are flexible and robust.

System Analysts job consists of:

- Gathering facts about the existing system
- Analyzing the basic methods and procedures of current information system
- Determining information needs
- Modifying, redesigning and integrating the existing procedures in the new system specifications to provide the needed information.
- System Analysis determines the design of the overall system
- Obtains the necessary technical help from programmers, from specialists and equipment engineers.

System Analysis follows the system through design, implementation, follow-up and re-evaluation

project that is always changing. Construction industry sources suggest that 85% of the project managers time is spent on communication and 70% of project documentation is paper based. So there comes need of ERP (enterprise resource planning) and Primavera to overcome the failure due to lack of management. ERP, PRIMAVERA set at the disposal of medium and large companies, all the necessary instruments that allow businesses to reach a new management level. PRIMAVERA ERP is a global management solution through which any company will be able to have the benefit of all the potentialities of an ERP used by thousands of companies and the experience of a team of experts. Project-oriented companies that want a total picture of their business are choosing to integrate their forward- looking, project management information with up-to-date financial details from their enterprise resource planning (ERP) system. Responding to this need for improved integration of the operational and financial aspects of businesses, it is necessary to provide business process integration between project management and ERP systems. The new integration solution will provide valuable insight into a company's forecasted cash and resource requirements, as well accurate performance information, for effective decision making. The resulting integration will provide these customers with a cost effective way to complement the strengths of ERP with best of breed project management from Primavera.

In an increasingly demanding and changing market, to conquer and keep a leadership positioning represents a challenge only at the reach of companies that are able to follow the natural evolution of the business world and adopt innovation as their banner. The actual patterns of competitiveness, linked to the constant need for high financial performances, impose the adoption of new business models based on advanced technologies that will allow superior levels of productivity. It is not enough for the operational and transactional processes of business activities to function perfectly, it is necessary the existence of proactive mechanisms of decision support that allow the business to be stretched far beyond the physical borders of the company. A product with a high level of extensibility and a great adequacy capacity that sets at disposal of any company all the mechanisms that allow its adjustment to any business. It is required to connect Primavera(R) project management and SAP(R) solutions for customers. The resulting integration will provide these customers with a cost-effective way to complement the strengths of SAP with best-of-breed project management from Primavera. With integration of ERP to Primavera, companies will be able to tie project events to supporting business processes, such as procurement, materials management and service requests, to improve relationships with customers and achieve the fastest time to market. For example, when key project milestones are achieved, the ERP software can trigger the creation of purchase orders in ERP for integrated materials management. Companies that are proactively managing costs, resources and schedules no longer need to struggle to make their project management and ERP systems successfully interact.

To study - ERP (SAP-systematic application product)) ,various implementation phases, benefits of using ERP in construction industry, problems associated with material management in construction industry, material management modules of ERP

1) To study-PRIMAVERA ,various features of PRIMAVERA ,benefits of using PRIMAVERA Identify benefits of using integration of ERP (HIT OFFICE) and PRIMAVERA for project management. Collection and study of literature pertaining to the dissertation work and studying need of integration

2) Studying PRIMAVERA and its various features, which will help to note the progress of project at any particular phase of project.

4)Working on ERP-material management module developed for small project, to tackle problem related to material management for same small project

5)Applying PRIMAVERA features for the same above small project.

6) To conclude from above work, benefits of integration of ERP and PRIMAVERA and then recommend for the same It is clear that a great deal of inefficiency exist in the mainly paper based process deployed predominantly in the construction industry, centralized digital information management is used to support project management and distribution of project information and to promote co-operation in building process. Need of Integration -Many organizations utilize Primavera project management software for scheduling purposes and ERP for cost management and material management. This creates a significant need to integrate the data between these two systems so the project stays synchronized. The systems need to be integrated so that project-related data can synchronized between the systems and to eliminate the costly and time-consuming data entry required keeping both systems consistent. Key ERP information, such as resource and material availability, needs to be provided to project managers working in Primavera to improve their effectiveness during project planning, scheduling, and execution activities. Scheduling, resources, material management and time information need to flow between the two systems to ensure that the project can be completed as planned.

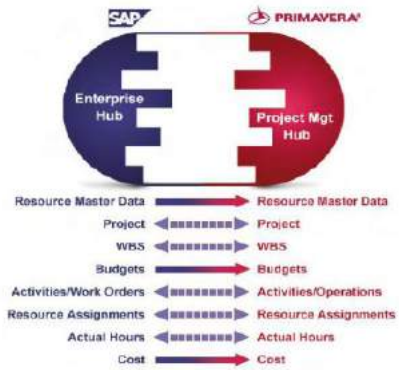

Fig.2.1 ERP information, Problems associated with materials management-

Since the materials management system described earlier involves several departments of the company, the co-ordination and communication becomes critical to the smoothness of the whole workflow. Unfortunately, all project managers agree that such co-ordination is difficult to achieve, and mis-communications often occur. Compounded by the variation orders initiated by the developers and the technical deficiencies of the suppliers problems and mistakes are often found in the process. According to the project managers, typical problems that occurred in the materials management processes are summarized below.

(1) Materials ordering **-** Abortive materials resulted from late notification of variations, Poor estimation of wastage level, Difficulty in tracing the outstanding amount of material .

(2) Delivery and distribution - Premature delivery causes damage of materials Late delivery results in progress delay ,Insufficient storage space for materials delivered to site, Insufficient delivery lead time allowed , Delivery timing and quantity does not match with progress ,Poor control of stockpile results in damaged, Poor control of materials check out ,Lack of appropriate materials handling equipment

(3) Payment -Payment date to suppliers not controlled by quantity surveyors, Inaccurate estimation of materials on-site ,Unit rates of variation items not agreed with suppliers The main reason for most of the problems quoted by the project managers is the lack of integration and sharing of materials information across various functional departments. For example, if progress review and reporting can be integrated into the materials ordering system, the chance of inappropriate delivery time can be avoided. Similarly, surplus materials can be reduced if there is continuous reconciliation of stockpile against work done and outstanding work. As

commented by some project managers, monetary loss from damages, surplus materials and the like is almost unavoidable despite close monitoring of the project (under the current system). Therefore, allowance is made in the pricing of work items during the tender stage to cover the potential loss. If the materials management system can keep the damages and wastage of materials to the minimum, savings can be transferred to the developers. **Module of ERP**

Sr.No.	Modules	Features
1	Materials Management	• Purchasing • Inventory management • Warehouse management • Invoice verification

FEATURES OF PRIMAVERA-

1) Centralized Project Repository

2) Enterprise Project Structure & Codes

3) Cross-project Analysis and Reporting

4)100% We-based

5) CPM Scheduling :

6)Float Path Analysis :

7)Cross-Project Dependencies

8)Projects Reports 9)Resource Assignments

10)ERP or Accounting Integration :

11)Resource Leveling :

12)Baseline Management-based

3.1 COMPANIES SURVEY:

High-rise and Hit Office are readymade ERP Software which are widely used by the construction companies. Companies targeted are –

 Microsoft projects is used by -

1) Goel Ganga Group, Pune.

2) Nyati Group, Pune.

3) Kohinoor Planet Constructions Pvt. Ltd., Mumbai.

4) Amit Enterprises Pune.

5) Rohan Construction Group, Pune

 Primavera is used by –

1) J.B constructions, Pune

2) Suratwala Housing Pvt. Ltd., Pune &

3) Dreams Construction Pvt Ltd, Pune.

4) Kumar Properties

5) Vedant Construction Pvt Ltd, Pune.

6) Puranic Developers Pune.

7) DSK Construction Pvt Ltd, Pune

8) J Kumar Infrastructure

While some of the companies are using tailor made ERP which is customized by the respective companies to suite their requirements. Following are some of the companies selected for the survey

 SAP is used by-

1) L&T Pawai, Mumbai.

2) City Corporation Limited, Pune.

3) Panchashil Pvt. Ltd. Pune.

 ORACLE is used by-

1) Akcruti City Ltd., Mumbai.

As per market survey which I conduct in pane& some part of Mumbai & due to the time limit I found following things which I mention with the help of graph

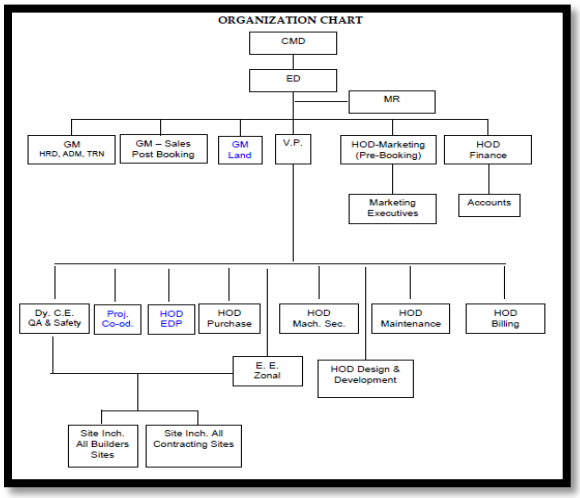

Fig.-3.2 Market survey analysis for software use

So as per survey which I conduct with various construction company, I am focus on" Primavera and Microsoft projects (MSP)" Software only .

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TECH-CHRONICLE
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Self Compacting Concrete (SCC) : A New Age Concrete

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ABSTRACT

This paper deals with the basic concept, methodology, advantages and limitations of Self Compacting Concrete (SCC) a concrete of new age. It is new age concrete because of its technological superiority over traditional or conventional vibrated concrete. It is technologically advanced due to its characteristics such as flow ability, passing ability and segregation resistance combined with altered rheology. Further, this paper discusses the various philosophies and approaches to mix design (proportioning) of SCC and so also the way SCC needs to be specified. The paper deliberates on various tests and test methods which are particular to SCC. The paper also discusses site requirements and preparation and formwork required for SCC, its casting and placing and the appearance and surface finish.

Key words: Self Compacting Concrete; flow ability; passing ability; segregation resistance; plastic viscosity.

Introduction:

With the rapid development of construction of mega structures the world over, and having the problems of congestion of reinforcement in principal structural members the demand for Self Compacting Concrete (SCC) application is increasing.. The design issues are further complicated due to the high risk of seismic zone, vulnerability to cyclonic storms and huge capacity addition of the plants. SCC has become the only choice in such difficult site environments. The important driving force behind the development of SCC is the development of concrete mix where placing and compaction has minimal dependence on the standard of workmanship available on a particular site which improves the true quality of the concrete in the final structure, and hence its durability.

SCC is considered as a breakthrough in concrete technology and greatest technological advancement and most revolutionary development in concrete technology over the years due to its improved performance and working environment. It has wide application from thin elements to bulk robust structures. SCC is a concrete of future, as it will be replacing normal concrete due to its distinct advantages.

SCC also called as Self Consolidating Concrete or Rheodynamic concrete is an innovative concrete that does not require vibration for placing and compaction. It is able to flow under its own weight, completely filling formwork and achieving full compaction, even in the presence of congested reinforcement. The hardened concrete is dense, homogeneous and has engineering properties at par with and durability as traditional vibrated concrete. The principle behind SCC is that the settlement of aggregates is related to the viscosity of the fresh concrete.

SCC can be produced using the same ingredients as that of normal concrete. The proportioning of SCC mix is much more scientific than that of conventional concrete mixes. SCC mix requires high powder content, lesser quantity of coarse aggregate, high range superplasticiser and Viscosity Modifying Agent (VMA) to give stability and fluidity to concrete mix. The workability of SCC is equilibrium of fluidity, deformability, filling ability and resistance to segregation. This equilibrium has to be maintained for a sufficient time period to allow for its transportation, placing and finishing. Combinations of tests are required to characterize the workability properties.

SCC was developed in late 1980s in Japan at University of Tokyo. In Europe it was probably first used in civil works for transportation networks in Sweden in mid 1990s. The EC funded a multinational, industry lead project "SCC" 1997-2000 and since then SCC has found increasing use in all European countries and the use is increasing all over the world, and in India also. In India, the SCC has been used in Kaiga Nuclear Power Project (Karnataka) and also in Kota Atomic Power project (Rajasthan). Some of the Indian projects where SCC has been used are Delhi Metro Project., Tarapore Atomic Power Project, Gosikhurd Project – 5000 Cum., Purna Dam Project and Lower Wardha Project – 2000 Cum. The use of SCC is increasing day by day in India and many infrastructure projects are going in for SCC, the example being ' The Signature Bridge' on river Yamuna near New Delhi and the Bandra-Worli sea link project, Mumbai and many mono rail and metro rail in India are being constructed by using SCC.

Definitions of SCC:

The Self compacting concrete has been defined by many. EFNARC, May 2005, Guidelines for SCC defines it as "Concrete that is able to flow and consolidate under its own weight, completely fill the formwork even in the presence of

dense reinforcement, whilst maintaining homogeneity and without the need for any additional compaction". An authority on SCC Khayat K. H. defines SCC as: "A highly flowable, yet stable concrete is one that can spread readily into place and fill the formwork without any consolidation and undergoing any significant seggregation", while Japan Concrete Institute defines SCC as "A concrete having self compatibility; self compactibility of concrete is its ability related to the placeability, with which it can be uniformly filled and compacted in the every corner of formwork by its own weight without vibration during placing."

Advantages of SCC:

SCC has many advantages, amongst them the main are the SCC mixes have good deformability enabling them to maintain homogeneity at fresh state. It can be placed & compacted under its self weight with little or no vibration effort & which is at the same time is cohesive enough to be handled without segregation & bleeding.

SCC offers a rapid rate of concrete placement with faster construction times & ease of flow around congested reinforcement. The fluidity & segregation resistance of SCC ensures a high level of homogeneity, minimal concrete voids & uniform concrete strength in situ, providing the potential for a superior level of finish & durability to the structure. Use of SCC in reduction of number of joints and homogeneous and dense concrete helps in reducing the leakages through the structure. SCC enables reduction in noise at site and so it ensures improved health and safety at site. The use of SCC reduces the exposure of the workers to sound intensities that are as low as one tenth of those produced when placing traditional vibrated concrete; introduction of SCC is truly a quiet revolution in concrete. SCC requires reduced manpower over conventional concrete (CC). Placing of SCC is much less strenuous activity than placing traditional vibrated concrete. More innovative designs, more complex shapes, thinner sections are possible with use of SCC. Reduced internal bleeding when SCC is used is responsible for a denser and stronger interfacial transition zone (itz) with respect to that of CC. The positive role of SCC in decreasing micro cracking and porosity of itz is also responsible for a more durable concrete. The denser microstructure of the itz in SCC may contribute for a lower plastic settlement, higher bond between steel and concrete matrix, lower permeability to oxygen and lower chloride diffusion coefficient with respect to corresponding values for conventional concretes. SCC allows rapid pumping of concrete. SCC has uniform, even surface with less surface defects, voids honeycombs etc. by virtue of good filling ability. SCC has improved aesthetics of final product with improved surface finish due to its good fluidity and deformability making SCC a competitive option vis-a-vas traditional concrete. A review of technical literature shows that SCC can flow (in formwork) horizontally a distance of 15-20m without segregation. A well designed SCC may have a free fall of as much as 8 m without segregation. However, from practical considerations height of free fall shall be restricted to ≤ 3 m and horizontal flow be as small as possible but restricted to 10 m.

Limitations of SCC:

SCC is a new technology and hence, requires well maintained and high degree of quality control & quality assurance methods. Production and placing of SCC need to be carried out by trained personnel only. Absence of internal and external bleeding in SCC however is one of the causes for their higher plastic shrinkage compared to traditional vibrated concrete. Hence, SCC should be cured as soon as practicable after placement to prevent surface shrinkage cracking. The lower MSA (nominal maximum size of aggregate) and reduction in % of coarse aggregate in volume of SCC, are responsible for lower modulus of elasticity compared to the conventional concrete. For this reason, the total shrinkage of SSC is also slightly higher. SCC requires good and leak proof formwork due to presence of more fines and flowable concrete. Special attention is needed in design of the formwork for pressures based on the flowability; cohesiveness, rate and method of placing or pumping (from top/from bottom) etc.Most common concrete mixers can be used for producing SCC. However, the mixing time may be longer than that for the conventional vibrated concrete. SCC is more sensitive to the total water content in the mix. Because of high cementitious content, the control on temperature of concrete is highly important in extreme hot environment. SCC places more stringent requirements on the selection of materials in comparison with CC. SCC mix requires a large number of trial batches. In addition to the laboratory trial batches, field size trial batches should be used to simulate the typical production conditions.

Engineering properties of SCC:

SCC and traditional vibrated concrete of similar compressive strength have comparable properties and hence SCC can be used in most applications where traditional vibrated concrete is used. However, SCC composition does differ from that of traditional concrete and the difference exists in the performance during fresh state; not much in terms of properties of hardened state. The most relevant structural properties of concrete are discussed here in short:

Compressive strength

SCC with a similar water cement or cement binder ratio will usually have a slightly higher strength compared with traditional vibrated concrete, and this is due to an improved interface between the aggregate and hardened paste due to absence of vibrations.

Tensile strength

For the SCC of specified strength, class and maturity, the tensile strength may be safely assumed to be the same as the one for a normal vibrated concrete. In fact due to improvement in homogeneity and denser microstructure, the tensile strength of SCC may be higher than for conventional concrete. Also, due to a less porous microstructure, tensile strength of SCC should be higher than that of CC.

Static modulus of elasticity

As SCC often has a higher paste content and lower maximum size of coarse aggregate and reduced coarse aggregate volume than traditional vibrated concrete, the *E*-value of SCC may be somewhat lower thereby lowering the tendency to form cracks with respect to CC with the same time dependent deformations.

Creep

Creep is defined as the gradual increase in deformation (strain) with time for a constant applied stress. It has two components, time dependent as well as stress dependant.

Due to the higher volume of cement paste, the creep coefficient for SCC may be expected to be higher than for normal concrete of equal strength, but such differences are small and covered by the safe assumptions in the tables and the formulae provided in the relevant codes.

Plastic Settlement

The beneficial effect in reducing internal bleeding is confirmed by the reduction of plastic settlement in SCC with respect to CC.

Plastic Shrinkage

Absence of internal and external bleeding in SCC, is one of the causes for its higher plastic shrinkage with respect to the CC. Researcher's have found that for the same loss of water, plastic shrinkage of SCC is at least two times higher than that of CC. As SCC might shrink more than normal concrete, use of synthetic fibers such as polypropylene/polyester will reduce shrinkage cracks being developed.

Drying Shrinkage

During hardening and in hardened concrete shrinkage is the sum of the autogenous and the drying shrinkage. In SCC a decrease in the maximum aggregate size results in a higher paste volume increases the drying shrinkage.

Bond to reinforcement, prestressing and wires

The bond strength between deformed bars and concrete matrix is higher in SCC than the CC. Higher bond strength in SCC is because of more stable paste capable of reducing the water film beneath the reinforcement, and consequently, to improve the mechanical properties.

Fire resistance

The fire resistance of SCC is similar to normal concrete. The use of polypropylene fibres in concrete has been shown to be effective in improving its resistance to spalling and bursting during fire. The mechanism is believed to be due to the fibres melting and being absorbed in the cement matrix. The fibre voids then provide expansion chambers and escape routes for steam, thus reducing the risk of spalling. Micro Polymeric (about 32 μm, Polypropylene) fibres have been successfully used with SCC.

Durability

SCC with the right properties will result in a material of consistently low and uniform permeability, offering less weak points for deleterious actions of the environment and, hence, better durability. The improved durability of SCC itself, as direct consequence of the better quality of the interfacial transition zone (itz) and of the lower tendency to crack in comparison with the CC.

Rheology of concrete

Rheology may be defined as the science of the deformation and flow of material and is concerned with relationships between stress, strain, rate of strain and time. The flow behaviour of fresh concrete does not conform to Newtonian liquids. The ratio of shear stress to shear rate is not constant for concrete. The fact that concrete can stand in a pile (as in case of slump test) suggests that there is some minimum stress necessary for flow to occur. The minimum stress is called as yield stress. Thus for fresh concrete the flow equation is expressed by Bingham equation which can be written as

$$\tau = \tau_0 + \mu \gamma'$$

Where τ = yield value indicating cohesion of material.

μ = Plastic viscosity

γ' = Rate of shear.

The rheology of fresh concrete like workability includes the parameters of stability, mobility and compatibility. These parameters encompass all the different attributes of fresh concrete which in turn determine the suitability of any concrete mix.

Specifying SCC:

SCC will normally be specified as a prescribed concrete or proprietary concrete. The proprietary concrete is the one wherein the producer assures the performance and need not declare the composition i.e. performance specification based concrete. The prescribed concrete is the one in which the user specifies the composition and the requirements. Basic requirements for SCC shall contain requirement confirming to "The European Guidelines for SCC" since Indian specifications are non existent, compressive strength exposure class(s) and/or limiting values of composition, e.g. maximum w/c ratio, minimum cement content. (as per *IS 456: 2000*, & it's Amendment no.3); maximum nominal coarse aggregate size; class of exposure related to chloride ion penetration, slump-flow class. In addition to the basic requirements the additional requirements and provisions shall be T_{500} value for the Slump flow test or a V-funnel class, L-box class, or segregation resistance class requirements. Specific requirements for SCC in the fresh state depend on the type of application, and especially on confinement conditions related to the concrete element geometry, and the quantity, type and location of reinforcement its congestion, inserts, cover and recesses etc; placing equipment (e.g. pump, direct from truck-mixer, skip, tremie) ; placing methods (e.g. number and position of delivery points); finishing method. The classifying system allows for an appropriate specification of SCC to cover these requirements, which are characterized by Flowability (Slump-flow), Viscosity (measure of the speed of flow); Passing ability (flow without blocking) ; Segregation resistance. The EFNARC specifications for SCC classify SCC for Slump-flow as SF1/SF2/SF3; Viscosity as VS1/VS2; Passing ability as PA1/PA2 and Segregation resistance as SR1/SR2.

Constituent materials for SCC:

The constituent materials for SCC are the same as those used in traditional vibrated concrete. However, mix

proportions for SCC differ from those of ordinary concrete, in that the former has more powder content and less coarse aggregate. Moreover, SCC incorporates high range water reducing admixtures (HRWRA, superplasticisers) in larger amounts and frequently a viscosity modifying agent (VMA) in small doses. Due to the fresh property requirements of SCC and higher powdery content, inert and pozzolanic/hydraulic additions are commonly used to improve and maintain the cohesion and segregation resistance. The addition will also regulate the cement content in order to reduce the heat of hydration and thermal shrinkage.

Among the various properties of aggregate, the important ones for SCC are the shape and gradation, rounded aggregates would provide a better flowability and less blocking potential for a given water-to-powder ratio, compared to angular and semi-rounded aggregates. Moreover, the presence of flaky and elongated particles may give rise to blocking problems. The influence of fine aggregates on the fresh properties of the SCC is significantly greater than that of coarse aggregate. Particles size fractions of less than 0.075 mm should be included in the fines content of the paste and should also be taken into account in calculating the water powder ratio.

The HRWRA helps in achieving excellent flow at low water contents and VMA reduces bleeding and improves the stability of the concrete mixture. The key function of a VMA is to modify the rheological properties of the cement paste. However, there exists the problem of incompatibility between cement and HRWRA, hence, before using any brand of cement it is advisable to verify its compatibility with the superplasticizer being used.

Mix composition – Mix design (proportioning) of SCC:

SCC is a concrete in which two normally incompatible properties of deformability and segregation resistance are both realized to achieve self compactibility by such measures as to increase the powder content and/or use of VMA. For this reason, a wide variety of formulas and methods are possible for SCC satisfying self compactibility requirements as well as performance requirements in hardened state such as durability.

Different mix proportioning methods can be grouped in two categories based on their approaches. In first category, the basic steps are determination of quantity of coarse aggregate then deriving approximate quality of mortar compatible for SCC mix. In second category, suitable mortar mix is first proportioned and then quantity of coarse aggregate is determined. The mixes proportioned by both the above categories can be further subdivided into 3 types as Powder type, VMA type, Mixed type (combined type).

A number of procedures for proportioning of mixes for SCC are there in vogue. These can be further classified into four methods as Empirical methods; Rheology based methods, Particle packing models and Statistical methods.

Typical range of SCC mix composition as per EFNARC is as under :

Constituent	Typical range by mass (kg / m^3)	Typical range by volume (liters / m^3)
Powder	380 - 600	- -
Paste	- -	300 - 380
Water	150 - 210	150 - 210
Coarse	750 - 1000	270 - 360
Fine aggregate (sand)	Content balances the volume of the other	
Water/Powder	- -	0.85 – 1.10

The mix proportioning for SCC is generally based on the methodology outlined below:

• evaluate the water demand and optimise the flow and stability of the paste
• determine the proportion of sand and the dose of admixture to give the required robustness
• test the sensitivity for small variations in quantities (the robustness)
• add an appropriate amount of coarse aggregate
• produce the fresh SCC in the laboratory mixer, perform the required tests
• test the properties of the SCC in the hardened state
• produce trial mixes in the plant mixer.

In the event that satisfactory performance is not obtained, consideration should be given to a fundamental redesign of the mix. Depending on the apparent problem, the following courses of action might be appropriate:

• adjust the cement to powder ratio and the water to powder ratio and test the flow and other properties of the paste
• try different types of addition (if available)
• adjust the proportions of the fine aggregate and the dosage of superplasticiser
• consider using or adjusting the dosage of a viscosity modifying agent to reduce sensitivity of the mix
• adjust the proportion or grading of the coarse aggregate.

Production for ready-mixed and site mixed SCC:

Self-compacting concrete is less tolerant to changes in constituent characteristics and batching variances than conventional vibrated concrete. It is also more sensitive to fluctuations in the total water content than conventional vibrated concrete. Accordingly, it is important that all aspects of the production and placing operations and methods right from control on quality of incoming materials are carefully supervised. Experience shows that SCC does not forgive any shortcuts. For production of SCC, the loading and mixing sequences are more important as it contain high amount of fine particles needed to be efficiently dispersed.

Experience has shown that the time necessary to achieve complete mixing of SCC may be longer than for normal concrete due to reduced frictional forces and to fully activate the super plasticiser. It is important that preliminary trials are carried out to ascertain the efficiency of individual mixers and the optimum sequence for addition of constituents.

Transportation and delivery:

SCC can be delivered either by truck mixer or truck agitator for long haul.

One of the main advantages of SCC is the increase in speed of placing. However, it is essential that the production capacity of the plant, journey time and placing capability at site are all balanced to ensure that site personnel can place the concrete without a break in supply and within the consistence retention time. Production stops can result in thixotropic gelling of concrete that has already been placed and this may affect the filling ability on restarting.

Site requirements and preparation:

Prior to delivery of the concrete, it must be ensured that appropriate site preparations have been made. These should include that the site can place the concrete at the agreed delivery rates, that acceptance procedures for the SCC are agreed and documented, that site personnel are trained in the specific requirements for placing SCC, that formwork is leakage free and properly prepared to receive SCC.

Sr. No.	Method	Property	Unit	Typical range of values.
1	Slump-flow by Abrams cone	Filling ability	mm	650-800
2	T500 mmslumpflow	Filling ability	sec	2-5
3	J-ring	Passing ability	mm	0-10
4	V-funnel	Filling ability	sec	6-12
5	V-funnel at T5minutes	Segregation resistance	sec	0-3
6	L-box	Passing ability	(h2 / h1) mm	0.8-1.0
7	U-box	Passing ability	(h2 - h1) mm	0-30
8	Fill box	Passing ability	%	90-100
9	Sieve stability test	Segregation resistance	%	0-15
10	Orpiment	Filling ability	sec	3-15

Formwork pressure and Formwork design:

Using SCC opens us a possibility of enhanced casting rates, as there are no longer any limitation due to the compaction work by the site staff. The negative side effect of speeding up the casting rate is the potentially high form pressures that might occur. The risk of high form pressures shall be considered at planning stage. Formwork pressure depends on the flowability and cohesion of the SCC, rate of pumping and vertical rise and the method of placing (from the top/from the bottom). Formwork design, including support and fixing systems, should assume that the full hydrostatic concrete pressure (of liquid of equivalent density) is applied to the formwork.

SCC can be placed in unusual or complex shapes which can be produced in formwork design that would normally not be possible with vibrated concrete SCC should be considered as a liquid and design of formwork should be done by calculating according to a hydrostatic pressure.

Placing and finishing on site and curing:

The process of casting/placing SCC shall be mechanized to a great extent. SCC is designed to have a very high flow combined with cohesion characteristics that ensure that the aggregate is uniformly suspended and does not segregate. The use of vibrators will affect this balance and will usually lead to significant segregation. For this reason, vibrating equipment should not be used with SCC except in the special circumstances. SCC is a liquid suspension following the rules of fluid mechanics while conventional vibrated concrete is a granular mass requiring vibration to be compacted. During placing, the concrete should be regularly visually checked to ensure that coarse aggregate is remaining at or very near the surface and that there is no indication of segregation. The concrete should form a regular advancing front at a shallow angle and be observed to flow round and fully enclose reinforcing bars without forming void pockets. There should not be excessive release of large air bubbles that would suggest air is being entrapped by the placing process. Foam on the upper surface is likely to indicate segregation. Check formwork for signs of leakage. SCC can be placed by direct discharge from truck mixers via a chute. Alternatively it can be first discharged into a skip (with tremie pipe) or to a pump. Pumping is the most common and suitable method of placing SCC due to high amount of fines and consequently the method from which most experience has been gathered. The usually high viscosity of SCC may require a slower pumping rate, in order to avoid high pressure built up in the piping system. High pressure may cause aggregate separation and pump stops. Pumping from the bottom of the formwork through a valve normally gives the best surface finish for any vertical element. It takes less air into concrete and allows faster casting rates than pumping from the top. SCC normally produces a very high quality finish giving a mirror copy of the formwork. The defects such as blowholes/bug holes, honeycombing, vertical sand stripes and other color variations, plastic or drying shrinkage cracking can be found in all types of concrete but with care, SCC can give an improved finish compared to traditional concrete

Curing is important for all concrete but especially so for the top-surface of elements made with SCC. These can dry quickly because of the increased quantity of paste, the low water/fines ratio and the lack of bleed water at the surface. As there is very little or no bleeding the concrete will be more sensitive to plastic shrinkage cracking. The tendency of plastic shrinkage increases with the increase in the volumes of fines. Initial curing should therefore commence as soon as practicable after placing and finishing in order to minimize the risk of surface crusting and shrinkage cracks caused by early age moisture evaporation.

Test methods of SCC:

The test methods for SCC has not yet been standardized, and the tests described are not perfected or definitive. So far no single test has achieved universal approval and most of them have their limitations. Similarly, no single method of test have been found which characterizes all the relevant workability aspects so each mix design shall be tested by more than one

test method for different workability parameters. They are mainly ad-hoc methods, which have been devised specifically for SCC. One principal difficulty in devising such tests is that they have to assess three distinct, though related, properties of fresh SCC – its filling ability (flowability), its passing ability (free from blocking at reinforcement), and its resistance to segregation (stability). There is no clear relation between test results and performance on site; there is little precise data, therefore no clear guidance on compliance limits; the test methods and values are stated for maximum aggregate size of up to 20 mm.

List of test methods for workability properties of SCC:

Some most frequently used tests are given in brief as below:

Test 1: Slump flow & Test 2: T_{500}time

The slump flow is used to assess the horizontal free flow of SCC in the absence of obstructions. It was first developed in Japan for use in assessment of underwater concrete. The test method is based on the test method for determining the slump. The diameter of the concrete circle is a measure for the filling ability of the concrete.

Test 3: V- funnel test

The test was developed in Japan and used by Ozawa et al. The equipment consists of a V-shaped funnel. The V-funnel test is used to determine the filling ability (flowability) of the concrete with a maximum aggregate size of 20mm. The funnel is filled with concrete and the time taken for it to flow through the apparatus measured.

Test 4: L- box

The test assesses the flow of the concrete, and also the extent to which it is subject to blocking by reinforcement. It assesses filling and passing ability of SCC, and serious lack of stability (segregation) can be detected visually.

Test 5: U- Tube

The test is used to measure the filling ability of self-compacting concrete. . It provides a good direct assessment of filling ability.

Conclusions:

From the above deliberations it can be concluded that - -

SCC offers a rapid rate of concrete placement, with faster construction times and ease of flow around congested reinforcement. The fluidity and segregation resistance of SCC ensures high level of homogeneity, minimal concrete voids and uniform concrete strength, providing the potential for a superior level of finish and durability to the structure.

The elimination of vibrating equipment improves the environment on and near construction sites where concrete is being placed, reducing the exposure of workers to noise and vibration.

The improved construction practice and performance, combined with the health and safety benefits, make SCC a very attractive solution and truly a new age concrete for both precast concrete and on-site civil engineering constructions.

Remarks: The case studies of SCC in Indian scenario and also the photographs on various aspects of SCC are not included in this paper for want of space.

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Earn Value Management a Boon to Indian Construction Industry

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***Abstract*— Earned Value Management (EVM) is a systematic approach to the integration and measurement of cost, schedule, and technical (scope) accomplishments on a project or task. It provides both the government and contractors the ability to examine detailed schedule information, critical program and technical milestones, and cost data. .EVM technique can do forecasting of duration for a project .It gives a well-defined pathway of existence to the project and when to take precautionary measures for updating the project if project is behind the schedule. It can be used to identify the cost overruns and time overruns. EVM technique can be applied using certain software's like MSP, Primavera, etc.**

Keywords—*Technical Milestones, EVM, Primavera, MSP.*

I. INTRODUCTION

Proper analysis of a project requires four major items: Budget, earned value, actual costs and forecasts. All four are needed to obtain a true picture of the project's health. If you analyze only the budget versus actual costs, an incorrect representation often results. For example, if the project spending is 10% under budget, this might appear as if the project is doing very well. However, when the project status or earned value is added to the analysis, it may then show that only half of the originally planned work has been performed. If so, we have a project that is behind schedule, and the completed work costs much more than originally planned.

This study can be applied at various levels of a WBS and can be helpful in tracking the progress of a project. It will also help in knowing the status of the project and help in taking appropriate action for the delay or over budget.

It allows projects to be managed better – on time, on budget. Earned Value Management System is not a specific system or tool set, but rather, a set of guidelines that guide a company's management control system. In the case of cost overrun, project management team may execute a value engineering program for cost reduction either reducing scope and quality in some sections of project or providing additional budget to cover overrun cost. Similarly, for time overrun case, the may plan some program such as fast tracking or time crashing for time reduction. Therefore, the role of EVM as well as correct and on time forecasting is very important to achieve project goals.

II. TERMINOLOGY

A. Planned Value (PV)

The planned value (PV), formerly called the budgeted cost of work scheduled (BCWS), also called the budget, is that portion of the approved total cost estimate planned to be spent on an activity during a given period.[1] $Planned\ Value = Physical\ Work + Approved\ Budget$ PV can be looked at in two ways:

1. Cumulative PV is the sum of the approved budget for activities scheduled to be performed to date.
2. Current PV is the approved budget for activities scheduled to be performed during a given period. This period could represent days, weeks, months, etc.

B. Actual cost (AC)

Actual cost (AC), formerly called actual cost of work performed (ACWP), is the total of direct and indirect costs incurred in accomplishing work on an activity during a given period.[1] It can be looked at in terms of cumulative and current.

1. Cumulative AC is the sum of the actual cost for activities performed to date.
2. Current AC is the actual costs of activities performed during a given period.

C. Earned Value (EV)

The earned value (EV), formerly called the budgeted cost of work performed (BCWP), is an estimate of the value of the physical work actually completed. EV is based on the original planned costs for the project or activity and the rate at which the team is completing work on the project or activity to date.

EV is the quantification of the "worth" of the work done to date. Earned Value (EV) tells you, in physical terms, what the project accomplished.[1] EV can be presented in a Cumulative and Current fashion.

1. Cumulative EV is the sum of the budget for the activities accomplished to Date.

2. Current EV is the sum of the budget for the activities accomplished in a given period.

D. Estimate at Completion (EAC)

The Estimate at Completion (EAC) is the actual cost to date plus an objective estimate of costs for remaining authorized work[1]. The most common is: $EAC = Actual\ Cost\ (AC) + Estimate\ to\ Complete\ (ETC)$ The Estimate to Complete (ETC) is the cost of completing the authorized remaining work.

E. Rate of Performance (RP)

Rate of performance (RP) is the ratio of actual work completed to the percentage of work planned to have been completed at any given time during the life of the project or activity.[1] For example, suppose the server installation was halfway completed by the end of week 1; the rate of performance would be 50% because by the end of week 1, the planned schedule reflects that the task should be 100% complete and only 50% of that work has been completed. The EV would thus be $5,000 after week 1 ($10,000*50%)

F. Schedule Variance (SV)

Schedule Variance is the comparison of amount of work performed during a given period of time to what was scheduled to be performed. It is calculated as follows $SV = EV - PV$ A negative schedule variance means the project is behind schedule i.e. it took longer than planned to perform the work.

G. Cost Variance (CV)

Cost Variance is thecomparison of the budgeted cost of work performed with actual cost.It is calculated as follows $CV = EV - AC$ A negative cost variance means the project is over budget i.e. performing the work cost more than planned.

H. Schedule Performance Index(SPI)

SPI can be used to estimate the projected time to complete the project. It is calculated as follows $SPI = EV\ /\ PV$ SPI = 1 means that project is on schedule. SPI < 1 means that project is behind schedule. SPI > 1 means that project is ahead of schedule.

I. Cost Performance Index(CPI)

CPI can be used to estimate the projected cost to complete the project based on performance to date. It is calculated as follows $CPI = EV\ /\ AC$ CPI = 1 means that the planned and actual costs are same. CPI < 1 means that project is under budget. CPI > 1 means that project is over budget.

III. Elements of Earned Value Management

EVM integrates three critical elements of project management: scope management, cost management, and time management. It requires the periodic monitoring of actual expenditures and the amount of work done (expressed in cost units). To determine cost performance, EVM compares how much we have spent to what we planned to have spent to do the work we have done. To determine time performance, it compares the amount of work done to the amount of work scheduled to be done. To make these comparisons, EVM calculates cost and schedule variances, along with performance indices for project performance management. Based on these results, it forecasts the date and cost of the project at completion and highlights the possible need for corrective action. EVM uses the following project parameters to evaluate project performance:

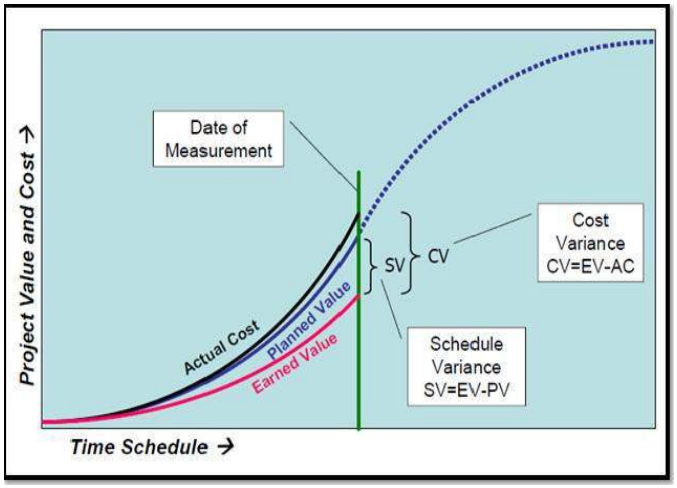

Figure 1: Earned value basics

Planned Value (PV): This is the cumulative planned cost for the work planned to be done on the project up to a given point in time. It is the approved budget for completing the work planned so far, and as such it is the cost baseline for the project. It was previously called the budgeted cost of work scheduled (BCWS).

Budget at Completion (BAC): This is the total amount of money expected to be spent on the project, and as such it is the value that PV is planned to reach at completion. Actual cost (AC): This is the cumulative actual cost spent on the project so far, including all accrued cost on the work done. AC was previously called the actual cost of work performed (ACWP).

Earned value (EV): This represents the cumulative amoun of work done up to a point in time, expressed in

cost units. It is expressed as the amount that was planned to have been spent on the work that has been completed up to this point. EV was previously called the budgeted cost of performed (BCWP). To calculate the EV for a given element
of work, the planned cost is multiplied by the percentage complete. The EV for the project is the sum of the EV for all the work elements. BAC, PV, AC and EV are expressed in cost units. That may be in units of actual money, in any currency. Or it can be expressed in hours or days of work done. PV, AC and EV can be calculated for any element of work.

DETAILS OF SOFTWARE Taking lead from the literature review the present study aims at evaluating Earned Value Analysis function of three software namely Microsoft Project 07, Primavera 6 and Develop Software. The following sections explain the software in brief.

A. M.S. Project 07(MSP)

Microsoft Project (or MSP or WinProj) is a project management software program which is designed to assist project managers in developing plans, assigning resources to tasks, tracking progress, managing budgets and analyzing workloads. The application creates critical path schedules, and critical chain and event chain methodology with third-party add-ons. Cost Variance and Schedule Variance are visualized in a Report.

B. Primavera 6

Primavera 6 manages and controls activities related to project management as well as portfolio management. Resources representing labour, materials and equipment are used to track time and costs for the project. Slippages of projects' activities are updated resulting in the adjustment of time related bars. It requires Data Base of Oracle My SQL.

C. Developed Software The Earned value analysis software developed in Visual studio 2008, SQL server and .NET (C#) language. And it provides robust project scheduling and management functionality. Features available are Planning, Scheduling, Cost Management and Project review.

CONCLUSION

There is a variety of challenges while planning Construction projects. When we are working to deliver high quality product on time and within budget constraints. Many of these challenges are related to a large degree of uncertainty, either in schedule duration, quality factors, or in design issues. By applying techniques that help quantify the nature of the uncertainty, separate the distribution of uncertainty in project schedule and insulate the project budget from the effects of the uncertainty, projects can be successful in gaining value from earned value analysis..In comparing Earned Value Management to Traditional Management, Traditional Management does not allow for analysis of the physical amount of work performed. Earned Value Management allows for both schedule and cost analysis against physical amount of work performed. The EVM approach deserves more attention in the construction field, where planning and estimation is needed.

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TECH-CHRONICLE
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SUSTAINABLE ELECTRICITY GENERATION FROM STAIRS FOR A GREEN BUILDING

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***Abstract* — In this modern world, we are using non-renewable energy sources such as petroleum as well as renewable sources like solar, wind, tidal power, etc. but still we couldn't overcome our power needs. So we have to generate electricity through each & every possible ways. Power can be generated through one unique way as well i.e. by the stepping of person on the stairs. The generated power will be stored & can be used for domestic purposes such as lighting. This system can be installed at homes, colleges, railway stations, multiplexes & shopping malls where people move around the clock. The utilization of waste energy of human foot power is very much relevant & important for populated countries like India & China. In this study, we will modify a normal stair-tread to move a small distance & the mechanical strain energy will be converted to electrical energy using Piezo-electric material. This arrangement will convert the foot power applied on stairs into efficient electricity which can be stored in batteries & can be utilized to operate LED lightings. This paper attempts to show how energy can be tapped & used at commonly used floor steps with the help of Piezo-electric materials. It's an eco-friendly, easily accessible & non-conventional power generation system which can turn any normal building into an Energy Efficient GREEN BUILDING.**

Keywords — Piezo-electric material, non-conventional power generation system, Energy Efficient GREEN BUILDING)

I. INTRODUCTION

Energy harvesting has been around for centuries in the form of windmills, watermills and passive solar power systems. In recent decades, technologies such as wind turbines, hydro-electric generators and solar panels have turned harvesting into a small but growing contributor to the world's energy needs. This technology offers two significant advantages over battery-powered solutions: virtually inexhaustible sources and little or no adverse environmental effects. With the need for portable and lightweight electronic devices on the rise, highly efficient power generation approaches are a necessity. The dependence on the battery as the only power source is putting an enormous burden in applications where either due to size, weight, safety or lifetime constraints, doing away with the battery is the only choice. Emerging applications like wireless micro-sensor networks, implantable medical electronics and tire-pressure sensor systems are examples of such a class. It is often impractical to operate these systems on a fixed energy source like a battery owing to the difficulty in replacing the battery. The ability to harvest ambient energy through energy scavenging technologies is necessary for battery-less operation. A 1 cm primary lithium battery has a typical energy storage capacity of 2800J. This can potentially supply an average electrical load of 100μW for close to a year but is insufficient for systems where battery replacement is not an easy option. The most common harvesters transducer solar, vibrational or thermal energy into electrical energy. The vibrational harvesters use one of three methods: electromagnetic (inductive), electrostatic (capacitive) or piezoelectric. The thermoelectric harvesters exploit temperature gradients to generate power. Most harvesters in practically usable forms can provide an output power of 10–100μW, setting a constraint on the average power that can be consumed by the load circuitry for self-powered operation.

Piezo electricity is the charge that accumulates in certain solid materials in response to applied mechanical stress. The word piezoelectricity means electricity resulting from pressure. It is derived from the Greek Piezo which means to squeeze or press, and electric or electron, which stands for amber, an ancient source of electric charge. Piezoelectricity is the direct result of the piezoelectric effect. The piezoelectric effect is understood as the linear electromechanical interaction between the mechanical and the electrical state in crystalline materials with no inversion symmetry. The piezoelectric effect is a reversible process in that materials exhibiting the direct piezoelectric effect also exhibit the reverse piezoelectric effect. For example, lead zirconate titanate crystals will generate measurable piezoelectricity when their static structure is deformed by about 0.1% of the original dimension. Conversely, those same crystals will change about 0.1% of their static dimension when an external electric field is applied to the material. Instead of looking for new ways to generate energy, we will be focusing on harvesting energy from everyday activities that would otherwise be lost. A person exerts lots of force when he walks down the stairs. The staircase power harvesting system intends to turn this energy into electrical power using a piezo-electric generator. In this study, which is a first in the literature, we propose an alternative solution to the dynamo and an improvement for the battery lifetime. We are proposing the use of piezoelectric generator, which is a clean and durable solution. Piezoelectric

generators employ active materials that generate a charge when mechanically activated. Today we see more and more applications using piezoelectric transducers. Their use as a source of electrical energy presents increasing interest for embarked electronic devices, low power consumption (less than 1 Watt) such as lamps based LED (Light- Emitting Diode), displays or sensors. Noticing that the stairs vibrate when someone steps on it, and that these vibrations are vectors of mechanical energy, we can recover and convert the mechanical energy contained in these vibrations into electrical energy by using electromechanical transducers, such as piezoelectric materials. The electrical energy thus produced can be used to power the lightings available for the stairs..

II. OBJECTIVE

The plan is to capture energy from the everyday motion of people traveling up and down a staircase. We can modify a normal stair tread to move a small distance and the vibrational energy will be converted to electrical energy using Piezo-electric Generator. From there, the energy will be stored in a battery for future use. The main goal is to harvest as much energy as possible, without compromising the reliability and safety of traditional stairs.

III. PIEZOELECTRIC MATERIAL

Many materials, both natural and man-made, exhibit piezoelectricity:

A. Naturally occurring crystals

- Berlinite ($AlPO_4$), a rare phosphate mineral that is structurally identical to quartz
- Sucrose (table sugar)
- Quartz
- Rochelle salt
- Topaz
- Tourmaline-group minerals

B. Other Natural Piezo Materials

- Bone: Dry bone exhibits some piezoelectric properties.
- Tendon
- Silk
- Wood due to piezoelectric texture
- Enamel
- Dentin
- DNA

C. Man-made crystals

- Gallium orthophosphate ($GaPO_4$), a quartz analogic crystal
- Langasite ($La_3Ga_5SiO_{14}$), a quartz analogic crystal
- The family of ceramics with perovskite or tungsten-bronze structures exhibits piezoelectricity:
- Barium titanate ($BaTiO_3$)—Barium titanate was the first piezoelectric ceramic discovered.
- Lead titanate ($PbTiO_3$)
- Lead zirconate titanate — more commonly known as PZT, Lead zirconate titanate is the most common piezoelectric ceramic in use today.
- Potassium niobate ($KNbO_3$)
- Lithium niobate ($LiNbO_3$)
- Lithium tantalate ($LiTaO_3$)
- Sodium tungstate (Na_2WO_3)
- Zinc oxide (Zn_2O_3)
- $Ba_2NaNb_5O_5$
- $Pb_2KNb_5O_5$

More recently, there is growing concern regarding the toxicity in lead-containing devices driven by the result of restriction of hazardous substances directive regulations. To address this concern, there has been resurgence in the compositional development of lead-free piezoelectric materials.

- Sodium potassium niobate ($(KNa)NbO_3$). In 2004, a group of Japanese researchers led by Yasuyoshi Saito discovered a sodium potassium niobate composition with properties close to those of PZT.
- Bismuth ferrite ($BiFeO_3$) is also a promising candidate for the replacement of lead-based ceramics.
- Sodium niobate $NaNbO_3$
- Bismuth titanate $Bi_4Ti_3O_{12}$
- Sodium bismuth titanate
- So far, neither the environmental impact nor the stability of supplying these substances have been confirmed.

D. Polymers

- Polyvinylidene fluoride (PVDF): PVDF exhibits piezoelectricity several times greater than quartz. Unlike ceramics, where the crystal structure of the material creates the piezoelectric effect, in polymers the intertwined long-chain molecules attract and repel each other when an electric field is applied.

IV. PIEZOELECTRIC GENERATOR PRINCIPLE

The conversion chain starts with a mechanical energy source: **Staircase**. Movement on the stairs produces vibrations and they are converted into electricity via piezoelectric element. The electricity produced is thereafter formatted by a static converter before supplying a storage system or the load (electrical device). In this study, before developing staircase piezoelectric generator, it was essential to begin with mechanical vibrations sources identification that means carrying out vibrations accelerations and frequencies measurement and analysis. So we have carried out measurement on an experimental Staircase to identify the situation where harvesting more energy is possible. We could then develop a piezoelectric generator adapted to the identified natural mode of vibration of the stair thread.

V. PROPOSED ELECTRICAL DESIGN

- The actual circuitry of the device is quite simple. The output of the Piezo Electric generator will be rectified and then sent to a small battery for storage. From here, the energy can be used in any form in which the user desires. It would be ideal to power LED lights which consume small amounts of power.
- For the battery we have chosen to use AAA sized NiMH batteries. They are safe and relatively easy to

work with. These batteries will be able to hold a significant amount of energy.

- The batteries can be charged quickly or slowly depending upon the current that is provided to them.

VI. PRODUCT FEATURES

- Electricity is generated through stepping on stair tread.
- Power can be stored for a future use.
- Design will be expandable to several steps and several Piezo-generators per step.
- Cost will be minimal to promote adoption

VII. APPLICATIONS

- Piezo Energy Harvesters can be embedded in shoes to recover walking energy.
- For ultra small wireless electronic devices.
- As a portable charger.
- Energy generated from piezo electric material can be used for charging mobiles, ipods, etc.
- Piezoelectric power can be created by putting a thin layer of material under a walkway which contracts and expands as people walk overtop it.
- One novel idea of piezoelectric power was to operate a remote control simply by pressing the buttons to generate the power necessary to send the IR signal

VIII. ADVANTAGES

- Harvest small, but still significant amounts of energy.
- An innovative approach to a device that people use every day.
- No compromise to safety or reliability.
- Marketing and appearance could encourage people to take the stairs instead of energy intensive alternative such as an elevator or escalator.
- Reduce dependency on battery power.
- Reduce installation costs. Self-powered wireless sensors do require wires, conduits and are very easy to install.
- Reduce maintenance costs. Energy harvesting allows the devices to function unattended and eliminates service visits to replace batteries.
- Provide long-term solutions. A reliable self-powered device will remain functional virtually as long as the ambient energy is available. Self-powered devices are perfectly suited for long-term applications looking at decades of monitoring.
- Reduce environmental impact. Energy harvesting can eliminate the need for millions on batteries and energy costs of battery replacements.

IX. RESULT

The results presented provide a platform to build off when using piezoelectric materials to charge batteries. Piezoelectric materials can be utilized for recharging batteries, brings power harvesting significantly closer to the commercial market and opens up many doors for its application. The rational for this comment revolves around the severe limitations that are brought on an electrical system when energy is stored in a capacitor. The major factor that really limits the electronics is the quick charge and discharge time of the capacitor; it can only be used to provide short bursts of power. This makes the use of computational electronics or data processing impossible. Additionally, the capacitor does not have a cell voltage that it maintains a constant voltage, but rather charges up to a high voltage then releases a quickly changing output, making the use of a voltage regulator, which dissipates energy, a necessity. Furthermore, portable electronics that are commercially available utilize batteries, allowing power harvesting systems that use rechargeable batteries to be easily adapted to current electronics. Power harvesting systems that utilize rechargeable batteries are the key to developing commercially viable self-powered electronic systems. Although the larger batteries will reach a charge level of 1.2 volts it is unknown without a charge controller how long the piezoelectric material would take to supply sufficient current for a full charge of these batteries to be achieved. It is apparent that both the PZT and Quick Pack are capable of recharging a discharged battery. When charging a battery, the most important electrical factor of the power supply is that it be able to provide a fairly significant amount of current.

X. CONCLUSION

The idea of power harvesting has become increasingly popular over the past few decades. With the advances in wireless technology and low power electronics, portable electronics and remote sensors are now part of our everyday lives. The key to replacing the finite power supplies used for these applications is the ability to capture the ambient energy surrounding the electronics.

Piezoelectric materials form a convenient method of capturing the vibration energy that is typically lost and converting it into usable electrical energy. This material has been used in the power harvesting field for some time; however, the energy generated by these materials is far too small for directly powering most electronic systems. This problem has been found by most all researchers that have investigated this field, thus showing the need for methods to accumulate the generated energy until a sufficient amount is present. Typically the storage medium used has been the capacitor, but the capacitor is not a good candidate because it can only provide short bursts of power. Realizing this issue showed that the rechargeable battery could be used with piezoelectric materials as an alternative to the capacitor.

XI. FUTURESCOPE

- In near future instead of using piezo crystals we can use piezo integrated tiles.
- Self energy generating cloths can also be used to create electricity.
- Design the special streets where generated electricity is used to charge electric cars.
- The total market for energy harvesting devices, including everything from wrist watches to wireless sensors will rise to over $5 billion in 2020.

- Electroactive polymers (EAPs) have been proposed for harvesting energy. These polymers have a large strain, elastic energy density, and high energy conversion efficiency. The total weight of systems based on EAPs is proposed to be significantly lower than those based on piezoelectric materials.
- Nanogenerators could provide a new way for powering devices without batteries. It only generates some dozen nanowatts, which is too low for any practical application.
- Noise harvesting NiPS Laboratory in Italy has recently proposed to harvest wide spectrum low scale vibrations via a nonlinear dynamical mechanism that can improve harvester efficiency up to a factor 4 compared to traditional linear harvesters.

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TECH-CHRONICLE
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COMPARISION OF DIFFERENT PROPERTIES OF SOIL WITH FIBERS AND WITHOUT FIBERS (PPF)

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Abstract - Abstract – The study is to find out the use of waste fiber materials in geotechnical applications and to find out the results .This will help to find out the effect of waste polypropylene fiber on the properties of soil like strength of fiber after comparing the results of soil with fiber and without fiber. The results are compared of soil with fiber and without fiber and its usability and cost effective approach has been checked out for further use of fiber as a reinforcement.

I. INRODUCTION

OVERVIEW:

For any land-based structure, the foundation is very important and has to be strong to support the entire structure. In order for the foundation to be strong, the soil around it plays a very critical role. So, to work with soils, we need to have proper knowledge about their properties and factors which affect their behavior. The process of soil stabilization helps to achieve the required properties in a soil needed for the construction work.

From the beginning of construction work, the necessity of enhancing soil properties has come to the light. Ancient civilizations of the Chinese, Romans and Incas utilized various methods to improve soil strength etc., some of these methods were so effective that their buildings and roads still exist. *In* India, the modern era of soil stabilization began in early 1970's, with a general shortage of petroleum and aggregates, it became necessary for the engineers to look at means to improve soil other than replacing the poor soil at the building site. Soil stabilization was used but due to the use of obsolete methods and also due to the absence of proper technique, soil stabilization lost favor. In recent times, with the increase in the demand for infrastructure, raw materials and fuel, soil stabilization has started to take a new shape. With the availability of better research, materials and equipment, it is emerging as a popular and cost-effective method for soil improvement. And for improvement of the soil a study on the properties of soil is necessary

Here, in this project, comparative study of soil has been done with the help of randomly distributed polypropylene fibers obtained from waste materials. The improvement in the shear strength, cohesiveness and consistency has been stressed upon and comparative studies have been carried out using
different properties of soil like specific gravity and Atterbergs limits.

OBEJECTIVE

1.To work out with various properties of soil.
2.To compare the Index properties of reinforced and unreinforced soil.
3.To investigate the use of waste fiber materials in geotechnical applications.
4.To evaluate the effects of waste polypropylene fibers on shear strength of unsaturated soil

II.METHODOLOGY

1. DEFINITION

Soil stabilization is the process of altering some soil properties by different methods, mechanical or chemical in order to produce an improved soil material which has all the desired engineering properties.

Soils are generally stabilized to increase their strength and durability or to prevent erosion and dust formation in soils. The

main aim is the creation of a soil material or system that will hold under the design use conditions and for the designed life of the engineering project. The properties of soil vary a great deal at different places or in certain cases even at one place; the success of soil stabilization depends on soil testing. Various methods are employed to stabilize soil and the method should be verified in the lab with the soil material before applying it on the field.

Principles of Soil Stabilization:

• Evaluating the soil properties of the area under consideration. • Deciding the property of soil which needs to be altered to get the design value and choose the effective and economical method for stabilization. Designing the Stabilized soil mix sample and testing it in the lab for intended stability and durability values.

Needs & advantages: Soil properties vary a great deal and construction of structures depends a lot on the bearing capacity of the soil, hence, we need to stabilize the soil which makes it easier to predict the load bearing capacity of the soil and even improve the load bearing capacity. The gradation of the soil is also a very important property to keep in mind while working with soils. The soils may be well-graded which is desirable as it has less number of voids or uniformly graded which though sounds stable but has more voids. Thus, it is better to mix different types of soils together to improve the soil strength properties. It is very expensive to replace the inferior soil entirely soil and hence, soil stabilization is the thing to look for in these cases.

 It improves the strength of the soil, thus, increasing the soil bearing capacity.

 It is more economical both in terms of cost and energy to increase the bearing capacity of the soil rather than going for deep foundation or raft foundation.

 It is also used to provide more stability to the soil in slopes or other such places.

 Sometimes soil stabilization is also used to prevent soil erosion or formation of dust, which is very useful especially in dry and arid weather.

 Stabilization is also done for soil water-proofing; this prevents water from entering into the soil and hence helps the soil from losing its strength. It helps in reducing the soil volume change due to change in temperature or moisture content.

 Stabilization improves the workability and the durability of the soil.

Methods:

1 Mechanical method of Stabilization:

In this procedure, soils of different gradations are mixed together to obtain the desired property in the soil. This may be done at the site or at some other place from where it can be transported easily. The final mixture is then compacted by the usual methods to get the required density.

2 Additive method of stabilization

It refers to the addition of manufactured products into the soil, which in proper quantities enhances the quality of the soil. Materials such as cement, lime, bitumen, fly ash etc. are used as chemical additives. Sometimes different fibers are also used as reinforcements in the soil. The addition of these fibers takes place by two methods;

- Oriented fiber reinforcement-

The fibers are arranged in some order and all the fibers are placed in the same orientation. The fibers are laid layer by layer in this type of orientation. Continuous fibers in the form of sheets, strips or bars etc. are used systematically in this type of arrangement.

- Random fiber reinforcement-

This arrangement has discrete fibers distributed randomly in the soil mass. The mixing is done until the soil and the reinforcement form a more or less homogeneous mixture. Materials used in this type of reinforcements are generally derived from paper, nylon, metals or other materials having varied physical properties. Randomly distributed fibers have some advantages over the systematically distributed fibers. Somehow this way of reinforcement is similar to addition of admixtures such as cement, lime etc. Besides being easy to add and mix, this method also offers strength isotropy, decreases chance of potential weak planes which occur in the oath

Soil properties:

1. Water content

Theory: The water content (w) of a soil sample is equal to the mass of water divided by mass of solid.

$$w= [(w2-w3)/ (w3-w1)]*100$$

W1= mass of empty container with lid.
W2=mass of the container with wet soil and lid.
W3=mass of the container with dry soil and lid.

2 .particle size distribution

Theory and Application:

Soil having particles larger than 0.075 mm sieve are termed as coarse grained soils. In this soil more than 50% of the total material by mass is larger than 75 micron coarse grained soil may have boulder, cobble,

Gravel, and sand coarse grained soil may have rounded to angular bulky hard rock particles with following sizes.

Boulder - more than 300mm dia

Cobble - Smaller than 300 mm larger than 80mm

Gravel - Smaller than 80mm and larger than 4.75mm
Coarse gravel - 20mm to 4.75 mm Course - 4.75 mm to 2 mm
Medium -2mm to 425 micron
Fine - 425 micron to 75 micron

3. Specific gravity:

Specific gravity of a substance denotes the number of times that substance is heavier than water. In simpler words we can define it as the ratio between the mass of any substance of a definite volume divided by mass of equal volume of water. In case of soils, specific gravity is the number of times the soil solids are heavier than equal volume of water. Different types of soil have different specific gravities, general range for specific gravity of soils:

Application:

1. Spacific gravity of a soil is an important property and it is used to determine void ratio, porosity, degree of saturation if density and water content are known.
2. Its value helps in identification and classification of soil.
3. It gives an idea about the suitability of soil as a construction material; higher value of specific gravity gives more strength for roads and foundation.

4. Atterberg limits:

1 Liquid limit:

It is the water content of the soil between the liquid state and plastic state of the soil. It can be defined as the minimum water content at which the soil, though in liquid state, shows small shearing strength against flowing. It is measured by the Casagrande's apparatus and is denoted by *wL*.

2 Plastic Limit:

This limit lies between the plastic and semi-solid state of the soil. It is determined by rolling out a thread of the soil on a flat surface which is non-porous. It is the minimum water content at which the soil just begins to crumble while rolling into a thread of approximately 3mm diameter. Plastic limit is denoted by *wP*.

3 Shrinkage Limit:

This limit is achieved when further loss of water from the soil does not reduce the volume of the soil. It can be more accurately defined as the lowest water content at which the soil can still be completely saturated. It is denoted by *wS*.

III .EXPERIMENTAL INVESTIGATIONS

A]

GW - Well graded gravel
GP - poorly graded gravel
SW - Well graded sand
SP - poorly graded sand

Experimental work:

The experimental work consists of the following steps:

1 Preparation of soil sample:

1. Determination of water content.
2. Partical size distribution.
3. Spacific gravity of soil without reinforcement.
4. Liquid Limit of soil without fibers.
5. Plastic Limit of soil without fibers
6. Shrinkage Limit of soil without fibers

B] **Preparation of reinforced soil samples.**

Sand	2.63-2.67
Silt	2.65-2.7
Clay and Silty clay	2.67-2.9
Organic soil	<2.0

1. Spacific gravity of soil without reinforcement.
2. Liquid Limit of soil without fibers.
3. Plastic Limit of soil without fibers
4. Shrinkage Limit of soil without fibers

- **materials**

1. Soil sample:

Location: Behind Suryodaya college of engineering, vihirgaon

2. Reinforcement:

Short PPF (polypropylene) fiber

C] Index parameter

Index and strength parameters of PP-fiber

Behavior parameters	Values
Fiber type	Single fiber
Unit weight	0.91 g/cm^3
Average diameter	0.034 mm
Average length	12 mm
Breaking tensile strength	350 MPa
Modulus of elasticity	3500 MPa
Fusion point	165 °C
Burning point	590 °C
Acid and alkali resistance	Very good
Dispersibility	Excellent

D] Preparation of samples

The value adopted in the present study for the percentage of fiber reinforcement is, 0.5% in the preparation of samples, if fiber is not used then, the air-dried soil was mixed with an amount of water that depends on the OMC of the soil.

If fiber reinforcement was used, the adopted content of fibers was first mixed into the air-dried soil in small increments by hand, making sure that all the fibers were mixed thoroughly, so that a fairly homogenous mixture is obtained, and then the required water was added.

IV. RESULTS

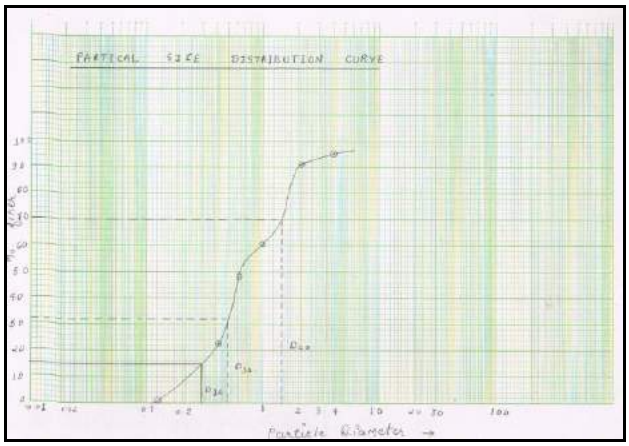
1. Water content

Water content of a soil:

Sr. no	Observation and calculation	Readings (grm)
1	Wt of container(W1)	40
2	Wt of empty container soil sample(W2)	110
3	Wt of container with dry soil lid(W3)	90
	Calculations :	
4	Water content = [(W2-W3)/(W3-W1)]*100	40%

2. Particle Size Distribution

Sr. no	Sieve no.	Mass of soil retained in germ	Cumulative mass of soil retained in germ	Cumulative %of soil retained in germ	% finer passing
1	80 mm				
2	40 mm				
3	20 mm				
4	10 mm				
5	4.75 mm	26	26	2.6%	94.79 %
6	2 mm	45.02	71.02	71	92.4
7	1 mm	320.01	391.03	39.1	60.8
8	600 mica.	110.7	501.73	50.17	48.83
9	475 mica	273.02	724.75	72.47	22.52
10	150 mica	2.45	999.31	99.31	0.47
11	75 mica	3.47	998..78	99.31	0.31
12	Pan	0.2	998.98	99.99	0.11



3. Specific gravity

- Specific gravity of soil without fibers:

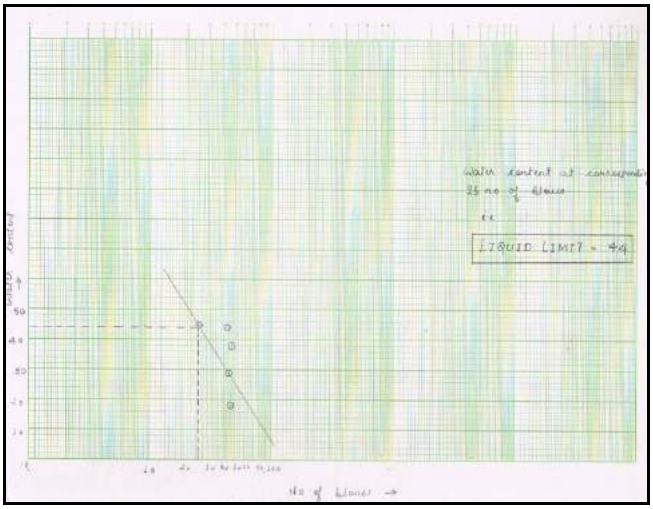
sample number	1	2	3
Mass of empty bottle (M1) in gems.	750	750	750
Mass of bottle+ dry soil (M2) in gems.	1250	1280	1260
Mass of bottle + dry soil + water (M3) in gems.	1960	1980	1970
Mass of bottle + water (M4) in gems.	1650	1650	1650
specific gravity	2.6315	2.65	2.6842
Avg. specific gravity	2.655		

4. Index properties

- Liquid Limit of soil without fibers:

Sr. no	Sample No.	1	2	3	4
1	No of blows	44	38	29	18
2	Container no.	2	4	1	5
3	Mass of Container + wet soil (germ)	31.53	30.29	36.24	21.22
4	Mass of Container + dry soil (germ)	27.05	26.59	30.14	18.25
5	Mass of water(3-4)	4.08	3.7	6.1	2.97
6	Mass of container (germ)	17.41	18.0	16.68	11.8
7	Mass of dry soil(4-6)	10.04	8059	13.46	6.45
8	Moisture content (5)/ (7)*100, (%)	40.33	47.01	45.31	46.04

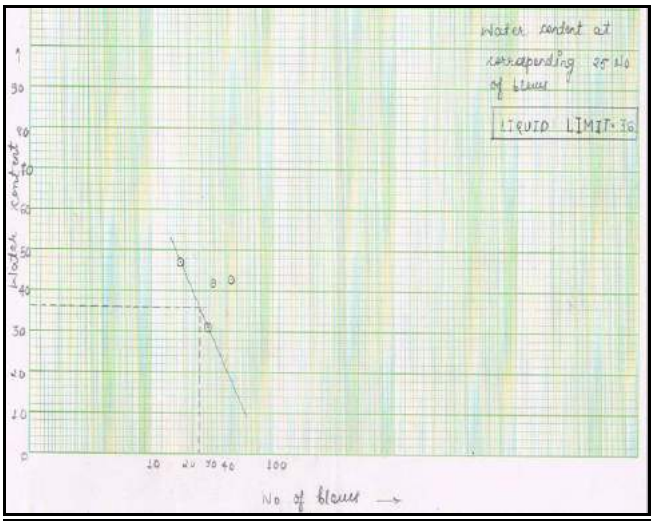
Liquid limit = 44



- Liquid Limit of soil with fibers:

Sr. No	Sample No.	1	2	3	4
1	No of blows	46	29	32	17
2	Container no.	2	4	1	5
3	Mass of Container + wet soil (grm)	30.2	25	35	29
4	Mass of Container + dry soil (grm)	26.3	22.2	29.6	24.65
5	Mass of water(3-4)	3.9	2.5	5.4	4.35
6	Mass of container (grm)	17.41	18.0	16.68	11.8
7	Mass of dry soil(4-6)	8.89	7.2	12.92	8.9
8	Moisture content (5)/ (7)*100, (%)	43.89	36	41.8	48.87

Liquid limit = 36



Plastic limit

- Plastic Limit of soil without fibers

Sr. no	Determination no	1	2	3
1	Container No.	2	3	4
2	Mass of container + wet soil(germ)	21.94	26.62	24.13
3	Mass of Container + dry soil (grm)	19.76	24.35	21.93
4	Mass of water (2)-(3)(grm)	2.1	2.27	2.2
5	Mass of Container(grm)	12.4	16.7	14.53
6	Mass of dry soil (3)-(5) (grm)	70.36	7.65	7.4
7	Plastic limit (4)/ (6)*100(%)	28.68	29.67	29.72
8	Average Plastic limit	29.35%		

- Plastic Limit of soil with fibers

Sr. no	Determination no	1	2	3
1	Container No.	4	2	3
2	Mass of container + wet soil(grm)	24.15	22.77	27.18
3	Mass of Container + dry soil (grm)	22.53	20.75	24.95
4	Mass of water (2)-(3)(grm)	1.62	2.35	2.23
5	Mass of Container(grm)	14.53	12.4	16.7
6	Mass of dry soil (3)-(5) (grm)	8	8.35	8.25
7	Plastic limit (4)/ (6)*100(%)	22.35	28.14	27.03
8	Average Plastic limit	25.8%		

Shrinkage Limit

- Shrinkage Limit of soil without fibers

Sr no	Determination no	1	2	3
1	Shrinkage dish no.	1	4	3
2	Mass of dish + wet soil pat (grm)	34.67	34.47	34.88
3	Mass of Dish + dry soil pat (grm)	26.28	26.13	26.23
4	Mass of water (2)-(3) (grm)	8.39	8.32	8.18
5	Mass of shrinkage dish empty (grm)	7.49	7.91	7.41
6	Mass of dry soil pat(Ws)=(3)-(4)	18.79	18.24	18.86
7	Initial water content (%)	44.65	45.61	43.3
8	Mass of weighing dish+mercury (filling shrinkage dish)	310	308	306
9	Mass of weighing dish empty	170	170	170
10	Mass of mercury (8)-(9) (grm)	140	138	136
11	Volume wet soil pat (cc)	10.21	10.14	10
12	Mass of weighing dish + displaced mercury (by dry pat)	212.2	210.4	210.6
13	Mass of mercury displaced (12)-(09)	41.2	40.4	4.6
14	Volume of dry soil packed	3.029	2.97	2.94

Shrinkage Limit = {[(M1-Ms)-(V1-V2)]*1000/Ms} = 43.61

- Shrinkage Limit of soil with fibers

Sr no	Determination no	1	2	3
1	Shrinkage dish no.	1	4	3
2	Mass of dish + wet soil	36	37	35.5

	pat (grm)			
3	Mass of Dish + dry soil pat (grm)	30.21	32.66	31.48
4	Mass of water (2)-(3) (grm)	5.79	4.34	4.02
5	Mass of shrinkage dish empty (grm)	7.49	7.91	7.41
6	Mass of dry soil pat(Ws)=(3)-(4)	24.42	28.32	27.46
7	Initial water content(%)			
8	Mass of weighing dish+mercury (filling shrinkage dish)			
9	Mass of weighing dish empty	170	170	170
10	Mass of mercury (8)-(9) (grm)			
11	Volume wet soil pat (cc)	11.2	12.21	12.75
12	Mass of weighing dish + displaced mercury (by dry pat)			

1. Based on Specific gravity of a soil- With mixing of 0.5% fibers (PPF) specific gravity of the soil increases by 0.3%. (From table no 3 and 4)

Strength of the soil is directly proportional to specific gravity, more is the specific gravity more will be the strength of soil.

2. Based on liquid limit of a soil - Soil without reinforcement and with reinforcement have liquid limit difference of 18.18%.

3. Based on plastic limit of a soil - As similar to liquid limit the plastic limit of soil is also reduces. It reduces from 29.35% to 25.8%.

% decrease in plastic limit is 12% (From table no 7 and 8), this result shows increase in shear strength, Cohesiveness and consistency of soil mass.

4. Based on Shrinkage limit of a soil - The value of the shrinkage limit in reinforced soil is less than that of unreinforced soil. Hence with the use of polypropylene fiber shrinkage reduces.

The value of shrinkage limit is used for understanding the swelling and shrinkage properties of cohesive soil. Lesser is the shrinkage more will the suitability of material for foundation, road and embankment as more will be the strength.

Shrinkage Limit = {[(M1-Ms)-(V1-V2)]*1000/Ms} = 38

On the basis of present experimental study, the following conclusions are drawn:

V.CONCLUSIONS

The value of shrinkage limit is used for understanding the swelling and shrinkage properties of cohesive soil. Lesser is the shrinkage more will the suitability of material for foundation, road and embankment as more will be the strength.

VI .REFFERENCES

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Earthquake Behaviour Of Structure With Shear Wall.

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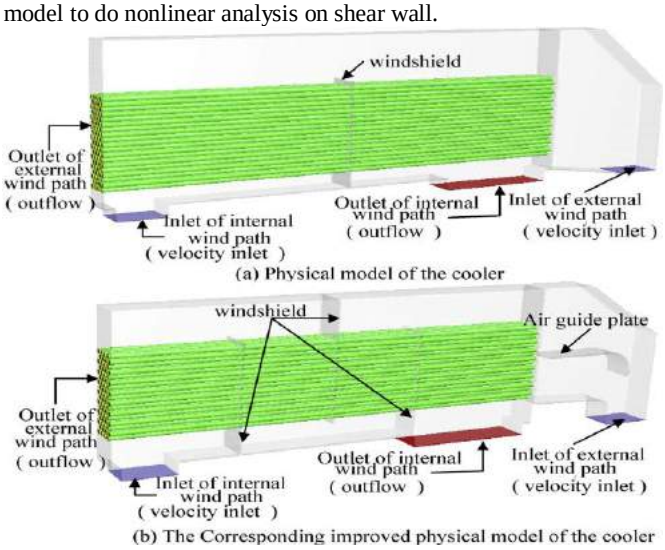
Abstract—Method of analysis to determine the influence of single shear walls (SSW) on the degree of coupling *Dock* and on the peak shear demand *PSD* for beams of coupled shear walls (CSW) in mixed shear wall structures (MSW). Non-coupled lateral load resisting structures such as singular planar walls will reduce primary bending moments in the coupled shear walls of MSW structures thereby increasing the degree of coupling. They will also change the location and magnitude of the maximum shear in and rotation of the coupling beams. These changes in the coupled wall bents may. Murray's law predicts that the flow ratio through the side branches scales with the ratio of the based on flow measurements performed by in angio graphically normal coronary arteries. The fit based on these measurements showed that the flow ratio through the side branches can best be described with a power of 2.27. The experimental data imply that Murray's law underestimates the flow through the side branches. shear wall goes by the name of a 'Composite Steel Plate Shear Wall' (CSPSW), which is used as the lateral resisting system in tall buildings. In this system, the steel plate buckles under medium-strong earthquakes, which may lead to instability. However, the buckling load of steel plates is usually a limited criterion for the design of CSPSW. This paper reports a series of experiments on CSPSW. The experiments were used to investigate the buckling load of a steel plate bolted to one side of a high strength reinforced concrete panel. Furthermore, theoretical modeling, based on energy methods, was used to obtain the elastic buckling coefficients of steel plates with various aspect ratios under shear loading.

Keywords-shear wall; Multi-Vertical-Line-Element Model; nonlinear; simulation

I.Introduction

Multiline computed tomography (MSCT) coronary angiography is a promising imaging technique capable of visualizing the coronary artery non-invasively. Although temporal and spatial resolution of the currently available MSCT equipment cannot match invasive technologies, MSCT is the best among noninvasive imaging techniques and radiation dose is now within limits such that serial imaging over time is acceptable. We expect that further developments in the near future result in increased resolutions such that 3D lumen reconstruction will be accurate enough for WSS computations. Since MSCT can image both the main branch and the side branches, it provides the geometrical data required to compute WSS near bifurcations. However, MSCT cannot provide the flow at the inlet or through the side branches; however, the proposed procedure has the advantages of accounting for complex mode shapes, non-uniform mass distribution, and interference effects from the surrounding. In addition, the technique allows for the contribution of higher modes. The combination of diameter and flow measurements in human coronary arteries in both mother and side-branches has only been reported once by In the current study, we use these measurements to establish the relation between flow and diameter, and diameter ratio and flow ratio. These relationships were used to determine inflow and outflow boundary conditions for CFD simulations. We will demonstrate the impact of prescribing these boundary conditions versus boundary conditions obtained from Murray's Law on the WSS distribution in patient-specific coronary bifurcations. For accurate the main difference about element stiffness matrix of Multi-Vertical Line-Element Model lies in the difference of considering shear deformation's magnitude. This text separates shear deformation from none pure bending deformation, only considering the shear deformation to participate in, reflecting the interaction of bending deformation and shear deformation to a certain degree. At last, utilizing existing reinforced concrete tensile-pressurized hysterias model and shear hysterias model to do nonlinear analysis on shear wall.



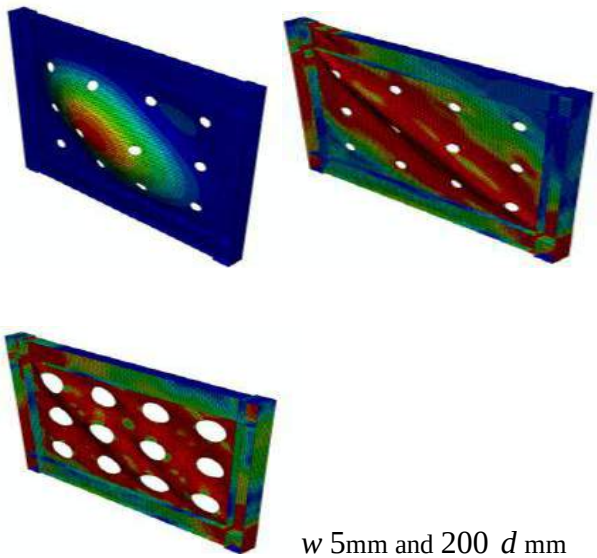
II. Literature

In contrast to hysteric loops of test and calculation, we can know that: the calculated yield load is close to value of test, but calculated yield displacement is less than normal; calculated limited load and displacement is more than value of test; Generally speaking, calculation and value of test are in well goodness of fit, in variety of axial hysterics relations, restriction point to 0.8Fsy, and the process depends on much assumption. On

the other hand, wind tunnel pressure measurements and finite element (FE) modeling of the structures are the effective alternatives for determining these responses.”.The shear panel configuration in this study is shown in Figure Canadian sections W310 x 118 and W530 x 82 were selected for columns and beams respectively. A regular pattern of perforations of diameter *c d* was formed at 1 meter centers. Ratio of perforation can be expressed by *d* / *S* where *S* is the width of imaginary diagonal strips shown. Web stiffener plates in columns were included for full beam- *E* was taken to be 200GPa with 1% post-yield stiffness. Yield stress of all elements was assumed to be 250MPa. Panel thickness of 5mm and 10mm (*h*= 2.97 and 3.53 respectively); and *dc/Sid* ranged from 0 to 0.35 were selected for investigation.

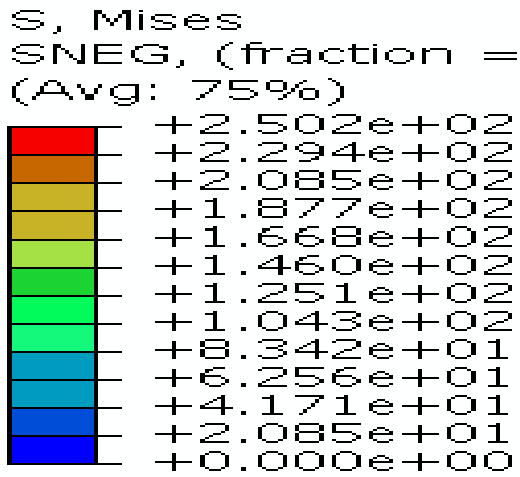
III. Method of study

Define abbreviations and acronyms the first time they are used in the text, even after they have been defined in the abstract. Abbreviations such as IEEE, SI, MKS, CGS, sc, dc, and rms do not have to be defined. Do not use abbreviations in the title or heads unless they are unavoidable.



w 5mm and 200 *d* mm

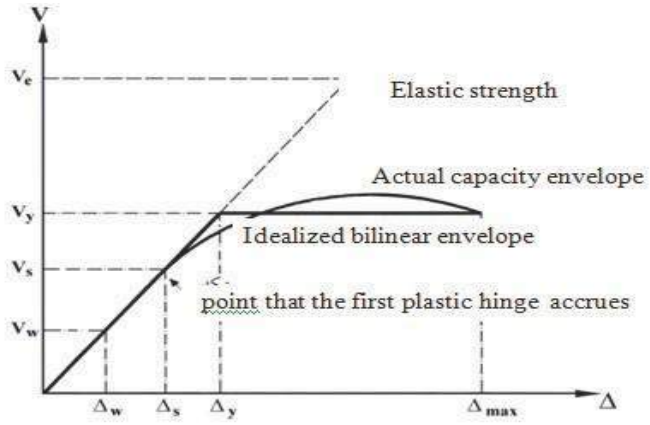
Buckled mode shapes and stress patterns at 1.4% drift for two selected models are shown in Figure 2. Perforations reduced eigenvalues but did not change the buckled shapes significantly. When *c d* was small, yielding of panel was limited to a middle diagonal strip. The remaining panel area was less effective. The boundary frames were too flexible to create a uniform tension field throughout the panel
area. As *c d* increased, yielding spread more uniformly. Furthermore, less deformation was observed in the top beam, indicating demand on the top beam was reduced



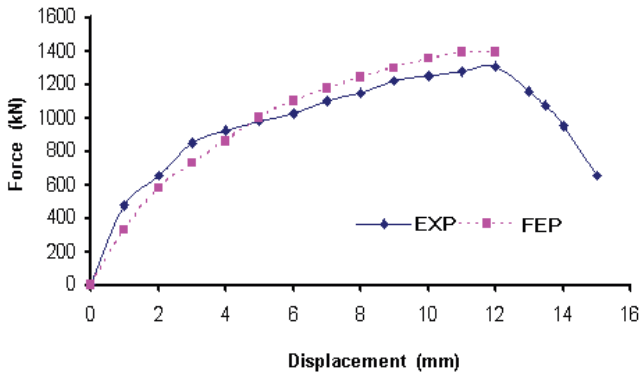
Normalized yield strengths and stiffness are related to perforation ratio in Figure As a distinct yield was not observed, yield strength was defined by the intersection of the load displacement curve with a line parallel to the initial slope of the curve and which intercepts the abscissa at 0.2% drift. Interestingly both strength and stiffness were reduced in a similar manner. To approximate effect of perforation, a linear reduction function is proposed herein. *VIPs* is the yield strength of perforated SPSW and *Vy* is that of a solid panel. Similarly *Kp* and *Ko* is stiffness for perforated and solid panel respectively. is a linear reduction coefficient. For the present SPSW configuration, *1.0* safely predicts the strength and stiffness. In contrast to hysteric loops of test and calculation, we can know that: the calculated yield load is close to value of test, but calculated yield displacement is less than normal; calculated limited load and displacement is more than value of test; Generally speaking, calculation and value of test are in well

IV.MIXED SHEAR WALL STRUCTURES

Adding single shear walls or cores to coupled shear wall structures or replacing coupled shear wall bents with single shear walls as shown in will alter the load distribution between the various lateral load resisting elements. A coupled wall structure has three characteristic structural parameters: bending stiffness *EI*, racking shear stiffness *GA* and axial bending stiffness *EAc*2 that define its behavior when subjected to horizontal loading. The lateral deflection behavior of a single shear wall can be well represented by its bending stiffness *EI*. Using the continuous medium method of analysis, it has been shown (Stafford Smith et al. 1982; Stafford Smith et al. 1984) that the horizontal deflections up the height of a high-rise mixed shear wall structure.
Four T shape shear walls are selected for investigating the influence of length and thickness of flange on the nonlinear behavior. Shear walls were analyzed by finite element program. All of T shape shear walls had a web with thickness of 100 mm and height of 2000 mm. Displacement of the flanged of T shape shear.

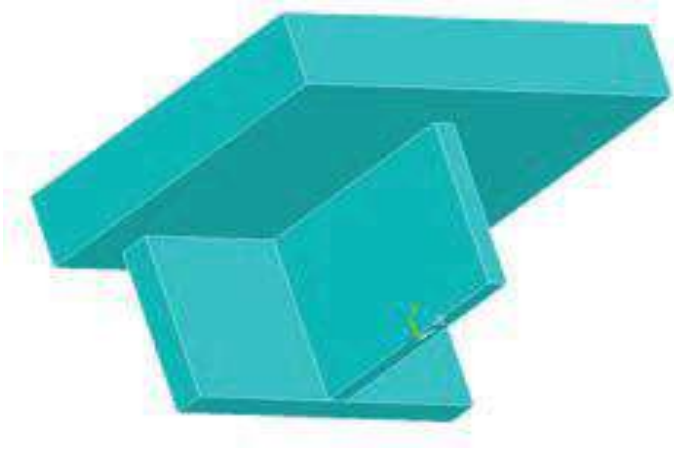


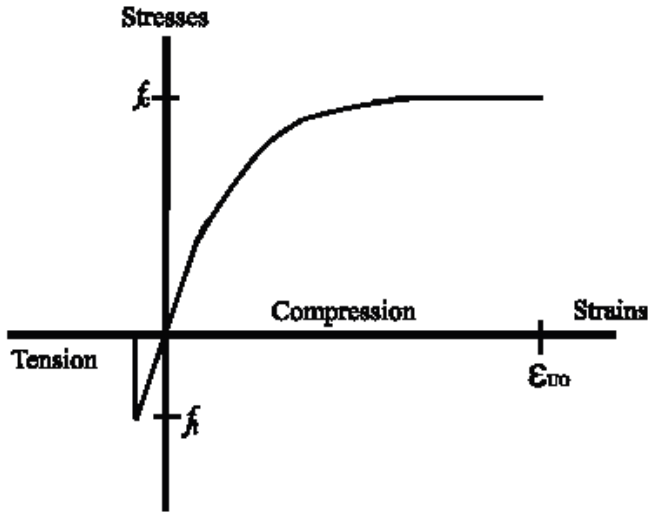
In order to investigate the accuracy of the nonlinear finite element program, results of I shaped shear wall were compared by finite element program. The capability of finite element program to reliably is high. The results of finite element program compared with experimental results to verify key aspects of the numerical model.



V. Results of Analysis

Shear walls were analyzed in each of four models. Results of analysis showed different responses in each models. First of model has 2500TSW. The analytical value of the first crack load is 232 kN. As the value at the failure is 1248 kN. This amount is maximum value between four models for failure. The displacement value at the ultimate force is 3.9 mm. Second model is 2700TSW.First of crack occur in 232 kN. So model is resisting and yielding in 668 kN. Load-displacement curve shows that the ultimate load is force of 1192 kN and in the 4 mm. In this model, ultimate load is 5 percent lower than the 2500TSW. From examination of plot in the Figure 4 it is obvious that in experimental analysis ultimate lateral strength is 1298 kN. This value is 1370 kN in finite element analysis which is 6 percent increase compared with results of experimental. In this paper, four different finite element models were generated to analytically predict





VI. DISCUSSION OF RESULTS

Results of analysis showed that flange shear wall with different flange dimensions had various response by lateral load. For instance, ultimate lateral load is 1248 kN in 2500TSW, 14 percent higher than that 3100TSW. Also, ductility is 4.5 in 2900TSW, which is %5 lower than 3100TSW. Since thickness of models is difference, responses are too. All of models cracked in lateral load about 232 kN. So flange dimension doesn't have been effect in flange shear wall. The load-displacement curve for four models indicated that yield force is very low in model with high thick. That means, 2500TSW has 125mm thickness, load of yielding is minimum. When thickness is decreasing in models, yielding force is increase. For example, increasing of 25 mm in thickness of flanged, 20 kN will decrease yield load and 1mm yield displacement. But after yield, shear walls had been various behaviours. Pu/ Py showed that in table These relations indicate how percent shear wall could be resistance lateral load before yield. For example, in 2700TSW, shear wall has been resistance lateral about 670 kN which is %56 of the ultimate load (1192 kN). In this model, 524 kN load is carrying

after yield. In the figure 7, amount of lateral load is showing before yield.

VII.CONCLUSION

The comparison of models indicates that finite element model used in this study is capable of predicting the nonlinear behavior of the models when these are different thickness. Results of analysis in four models and load- displacement of them indicated that 2500TSW had better behavior. It had been resisted about 1248 kN. This load is 5, 14 percent higher than other models. Also, ductility of this model showed a good agreement. For example, ductility in the 2500TSW model is 4.58 which is three percent higher than 3100TSW. The finite element models of four T shape shear wall in this study could increase shear absorb when were useful flange shear wall with more thickness. Results of analysis showed that 3100TSW had better strength after yield. This equal was 18% higher than 2500TSW. Crack pattern in all.

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TECH-CHRONICLE
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Investigation in Behaviour of G+7 Storied Building with the Introduction of Distinct Shaped Columns

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Abstract— **In the construction of buildings; square, rectangular and circular shape of column are commonly and conventionally used. In the construction of the building work, it happens that, two walls get crossed or makes T shapes or L shapes at any point or usually at the corners of building. Usually Structural members subjected to axial load and biaxial bending is en- countered in design practice from time to time; a typical example is the corner column in a framed structure. In recent years, the idea of using irregularly shaped columns such as L-shaped columns at the corners of framed structures and at the enclosure of elevator shafts has drawn the attention of investigators. It is felt that, there is no code provisions and available data for the determination of both strength and ductility of such biaxially loaded reinforced concrete columns.**

This paper lays a special emphasis on L, T and + shape columns, as the design of such columns can be performed in the future. This deals with comparison of forces and displacement in selected few columns in the structure with conventional rectangular shape columns and a structure with L, T and + shape columns through different graphs.

Keywords—Construction, axial load and biaxial bending, forces and displacement

I. INTRODUCTION

In the construction of buildings; square, rectangular and circular shape of column are commonly and conventionally used. In the construction of the building work it happens that two walls get crossed or makes T shapes or L shapes at any point or usually at the corners of building. The provision of insertion of different shapes of column in the intersection of such corners can be show better result than that of conventional shapes. It may undergoes there miscellaneous changes in column buckling, maximum bending moments, shear forces and changes in stiffness of structures. It is felt that current code provisions and available methods do not offer an insight into the determination of both strength and ductility of such biaxially loaded reinforced concrete columns.

The primary objective of this project was to study strength and deformational behavior of L-shaped and cross shaped tied columns under combined biaxial bending. This study lays a special emphasis on L and + shaped columns, as the design of such columns can be performed in the future. By introducing the L and cross shape column sections at necessary points in the structure can be analyse the effect on beam members. After then, the column section which will comes with optimum results will get designed for detailing for maximum bending moment and shear values as per the IS 456: 2000, IS 13920:1993 and other relevant codes.

II. A COMPARISON OF SEISMIC BEHAVIOR BETWEEN SPECIALLY SHAPED COLUMN FRAME STRUCTURE AND RECTANGULAR COLUMN FRAME STRUCTURES

According to the current Chinese code and technical specification, some frame structures with rectangular columns and specially shaped columns are designed respectively based on the criterion of the same section area, moment of inertia, initial stiffness of the specially shaped frame structure.

This paper analyzes the nonlinear seismic behaviors of the RC frame with specially shaped columns and RC frames with conventional rectangular columns under the unilateral horizontal ground excitations. But we can also analyze it for vertical dead load (under gravity) forces on the responses of the structures, the crack and yield rules of the beam and column elements are compared.

III. BIAXIALLY LOADED L-SHAPED REINFORCED CONCRETE COLUMNS

As per this paper, the results of an experimental and analytical investigation on the strength and deformation of biaxially loaded short and tied columns with L-shaped cross section are presented. The study explores the behavior of reinforced concrete columns under loads monotonically up to failure. A few tests loaded cyclically are also compared with those loaded monotonically. The strength interaction curves and load contours of L-shaped columns based on analysis and it provide advice for design information.

IV. L-SHAPED COLUMN DESIGN FOR BIAXIAL ECCENTRICITY

Rational shape for the load contours in L-shaped columns is proposed using the inverse method of analysis. Determination of the ultimate loads from the appropriate interaction curves derived from the shape of load contours is shown. An alternate and simple method based on the concept of an equivalent square or rectangular column is proposed to determine the theoretical ultimate loads using the interaction diagrams in square or rectangular columns. Theoretical loads computed by the two proposed methods are compared with the test results of 45 columns.

A. Buckling of Variable Cross-Section Columns: Integral-Equation Approach

A semianalytical procedure is presented for the axial buckling of elastic columns with step-varying profiles. Profiles with continuous variations can be approximated, to any desired degree of accuracy, by a series of step variations. Formulas for buckling loads for members with variable profiles and different boundary conditions can be obtained in terms of the section and profile parameters. The example problems present the solution of some common columns with variable cross sections. The procedure can be used to derive the elastic and geometric stiffness matrices for beam columns with variable cross sections as well as for other elements.

B. Column Behavior Of Cold-Formed Hollow Sections

The results of an experimental investigation into the strength and collapse behavior of Australian-produced cold-formed square and rectangular hollow section columns are described. The individual plate strengths calculated from the stub column tests on both square and rectangular hollow sections are compared with predictions based on the effective width formulae. Where in the section studied the local buckling and yielding occur almost simultaneously, leading to rapid load shedding after ultimate. Theoretical models of the postultimate collapse behavior, based on local plastic mechanisms, are summarized in the paper. The theoretical models are compared with the collapse behavior of the stub columns and long columns.

V. INITIAL POSTCRITICAL BEHAVIOR OF THINWALLED ANGLE SECTION COLUMNS

Angle section members are widely used as structural components to carry axial or transverse loads (Madugula and Kennedy 1985). A recent comprehensive review of theoretical and experimental work on buckling of angle members (Kennedy and Madugula 1982) shows that instability may occur in flexural, torsional-flexural, local plate, or torsional-flexural-local modes. The present work is concerned with the elastic flexural buckling of an equal-leg 90° angle member under concentric axial load, taking into account the influence of the deformation of the cross section as the load is increased after the onset of buckling. Previous studies (Goldberg et al. 1964; Studnika 1980) concentrated on the calculation of critical loads, while the emphasis here is on the initial postbuckling behavior of the column.

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Wind load analysis of buildings in hill-shape zone

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***Abstract*—wind load as a natural phenomenon on structures, especially tall buildings cannot be ignored. The magnitude of wind velocity alters significantly for equivalent structures located at various locations due to the changes in local roughness. The building is posed on top of hill-shape zone. A model of the structure was created in SAP2000 software to validate results of manual calculations based on two codes. it is shown that wind shear force applied on the building in three different heights Moreover, the distribution of wind load along the height of building based on two codes is different. IBC 2000 considers more details about topographic features of area and presents more accurate distribution of wind load along the height of building.**

Keywords—Static Wind load analysis, distributed horizontal load

1. INTRODUCTION

The wind velocity increases from minimum at ground level to a highest value at top of the building. Structure's shape obstacles to wind stream and form a positive pressure zone on the windward A growth in wind speed happen where the two zones face and the wind stream is transferred from the positive side to the negative. The wind force is the greatest significant parameter that affects the modelling of high-rise structures. Buildings over 10 stores would generally demand extra lateral displacement resistance system. The accurate loads happen throughout the life of the structure cannot be predicted. Most international building codes recognize several parameters in accordance with the boundary conditions of any buildings measured in the analysis to prepare for life safety. Improved structural design systems and construction methods as well as enhanced structural material strengths have enabled construction of increasingly tall buildings. In addition, with the development of modern cities, a large number of tall buildings are being constructed within small areas.

I. Selected structure

The selected structure is assumed to be located in Nagpur with the wind speed of 90 mph based on codes. The building is a flat roof with the plan view shown in the structural system of this building is a steel moment resisting frame. Figure 2 displays three views of structure in three different heights which have been modelled in SAP2000 software Due to the application of the building as a hospital, it is needed a high importance factor. This building has been modelled in SAP2000 software in three different heights, separately (low-rise, middle-rise and high-rise). A slab with the thickness of 10cm covers floors of each level. The thickness of walls all along the height of the building is 0.2m. All beam-to-column connections are fixed. The steel frame consists of fourteen columns that all have box-shape cross sections. All beams have I-shape cross sections. numerically in two perpendicular directions, X and Y and the results were compared with each other. Moreover, the obtained results from calculations have been compared with the results from the software.

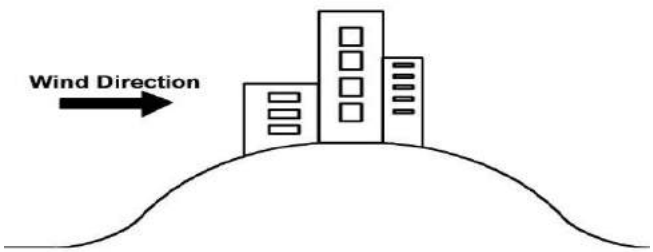


Figure. 1

II.METHODOLOGY

The wind can be supposed to blow in horizontal direction and followed the code of practice. No reduction in wind pressure shall be taken for the shielding effect of adjacent structures. Effect of wind on forward and backward to the wall is pressure and suction action types, respectively. On top of that, the effect of wind on the side

wall is considered to be a suction action type. Figure 3 illustrates these types of wind loads on a structure.

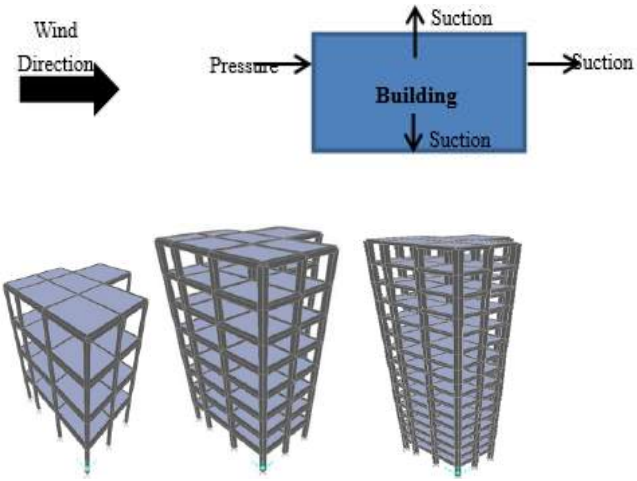


Figure 2. Three Types of Buildings Modeled in SAP2000 (a)Low-rise (b)Mid-rise (c) High-rise

Structural Code:

Basic Wind Speed: 90 mph Wind Stagnation Pressure: qs= 20.8 psf Importance Factor: Iw= 1.15 Expresser: C Combined height, expresser and gust factor coefficient: Ce Pressure coefficient for the structure or portion of structure Cq Design Wind Pressure: P Wind Pressure P= Cq.Ce.qs.Iw Based on the code, the parameter Ce and Cq for each element depend on height and position of buildings.For instance, figure 4 shows the first storey plan (h=4.6m). The wind is in x-direction, so the column (2-A), for example, is in windward position and the pressure of the mentioned column is calculated as follows.

qs = 20.8 psf = 0.996 kpa

Iw = 1.15

Ce =1.06

Cq = 0.8 (inward)

P= Cq.Ce.qs.Iw= 0.971 kpa

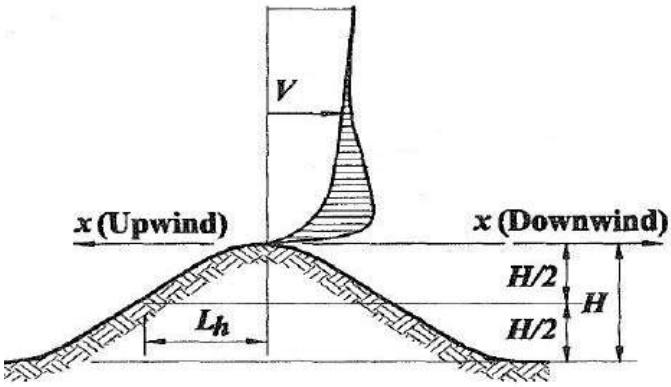
Leff = 4.5 m

h = 4.6 m

And to calculate the force at mentioned column:

F= P.Leff.h =20.1 kN

Leff is the effective width of column and is equal to sum of half span from both sides. The same procedure was applied to total circumferential columns in both X and Y directions from ground level to top for low-rise, mid-rise and high-rise structural models.



Following data is the wind pressure calculation for the column (2-A) of Mid-Rise building with a height of 32.5m (wind direction is in X-direction).

q: 1.867 (kpa)

q: 1.580 (kpa)

G: 0.871

Cp: 0.800 (windward)

GCpi: 0.55

P: 2.163 (kpa)

F = P.L .h = 44.8 *eff*

III.Result and discussion.

- In this study, wind force was applied to each column in both X and Y directions based on two mentioned codes. The whole forces from circumferential columns with their directions were combined and the absolute wind shear excited on the building was calculated for three different heights. Moreover, the results of SAP2000 models for three different heights are compared with both codes' results. Table 1 compare manual and software result for both codes.

		(Manually) (kN)	SAP2000) (kN)
Low-Rise	X-direction	17.09	15.6
	Y-direction	15.314	14.5
Mid-Rise	X-direction	33.473	31.42
	Y-direction	30.326	28.5
High-Rise	X-direction	67	62.3
	Y-direction	60.6	57.1

The distribution of wind force is different along the column's height; it starts from a zero value at ground level, increases dramatically to reach the maximum value at level 4.5m and from that point to the top of the building value of wind force goes down slightly along the height. The difference

between two structural codes is due to the topographic effect of the area. As mentioned in assumption part of this study and the building is posed on the hill shape ground. In fact, the wind force from the bottom of the hill tends to move, but hill works as a dam and prevents it from moving. So wind force becomes accumulated, goes along the height of the hill and when reaches the ridge suddenly releases.

IV.Conclusions

In this papers study, wind shear force for one steel structural building with the application of a hospital was obtained based on two structural codes IS 875 part-III also the structure was modelled in SAP2000 software and results were compared. The mentioned building was assessed in three different heights as Low-Rise, Mid-Rise and High-Rise. Based on achieved results, it can be clearly concluded that the effect of wind on the structure becomes more severe as its height increases. Moreover, it was shown As the building in this research is assumed to be built on top of the hill shape ground, distribution of wind force along the height of columns based on the two codes are different.

a.

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"Study and Analysis of P-Delta Effects on Building Structures with respect to Linear Static analysis"

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***Abstract*— In the traditional first order analysis or linear static analysis of structures, the effects of change in the structure actions due to structure deformations are neglected. However, when a structure deforms, the applied loads may cause additional actions in the structure that are called second order or P-Delta effects. Engineers have been aware of the P-Delta for many years. However, it is only relatively recently that the computational power aided to provide analytical approximations to this effect, which has become widely available. It is an engineer's judgment as to how accurately the second order effect needs to be accounted for in determining design forces and moments. In present study Seismic analysis of a multi-storey RC building with and without P-Delta effects is analysed by using STAAD structural analysis software. The building models with and without P-Delta effects have been analysed to investigate the maximum response in buildings in terms of displacements, column moment, beam moment, column shear and beam shear.**

***Keywords*— *P-Delta, structure deformation,* Seismic analysis, STAAD structural analysis software.**

I. INTRODUCTION

When horizontal loading acts on a building, causing it to deflect, the resulting eccentricity of the gravity loading from the inclined axes of the structure's vertical members causes the lateral displacements of the structure and the moments in the members to increase. This second-order effect is termed the P-delta effect. In heavily clad low- and medium-rise structures, the P-delta effects are small enough to be neglected. However, with the trend toward taller and lightly clad buildings with greater lateral flexibility, the P-delta effects become more significant. In the modern era the demands of high-rise are greater than earlier due to the provision that the number of satisfactory people can be accommodated in that but the inappropriate design may lead to catastrophic demolition or destruction of the structure which is obvious from the earlier few decades. In some cases, the P-delta effects are large enough to require an increase in the designed member sizes. In an extreme case of a very flexible structure with a large gravity loading, the P-delta effects could, if not accounted for, be severe enough to initiate collapse. Thus, in the design of any high-rise building, it is important to assess whether these second-order effects are significant and, if so, to account for them in the analysis and design. In addition the trend towards more slender and lighter building structures has resulted in potentially more significant P-delta effects; this has led to the demand for simple and accurate methods of P-delta analysis. Thus the aim of study is to recognize in what way the P Delta effects influence the variation of responses of structure such as bending moments, displacements and shear forces against linear static analysis.

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II. METHODOLOGY

The various methods used for P-delta analysis are

i) Direct P-Delta Method.
ii) Negative Bracing Member Method.
iii) Iterative P-Delta Method.

Out of these Iterative P-Delta Method is used in this study. The iterative P-delta method is based on the simple idea of correcting first-order displacements, by adding the P-delta shears to the applied story shears. Since P-delta effects are cumulative in nature, this correction and subsequent reanalysis should be performed iteratively until convergence is achieved. At each cycle of iteration a modified set of story shears are defined as:

$$\Sigma V_i = \Sigma V_1 + (\Sigma P)_{\Delta i-1} / h \qquad (1)$$

Where ΣV_i is the modified story shear at the end of ith cycle of iteration, ΣV_1 is the first-order story shear, ΣP is the sum of all gravity forces acting on and above the floor level under consideration, $\Delta i-1$ is the story drift as obtained from first-order analysis in the previous cycle of iteration, and h is the story height for the floor level under consideration. Iteration may be terminated when

$$\Sigma V_i \approx \Sigma V_{i-1}$$
Or
$$\Delta_i \approx \Delta_{i-1} \qquad (2)$$

Generally for elastic structures of reasonable stiffness, convergence will be achieved within one or two cycles of iteration. One should note that since the lateral forces are

being modified to approximate the P-delta effect, the column shears obtained will be slightly in error. This is true for all approximate methods which use sway forces to approximate the P-delta effect.

III. MODEL DISCRIPTION

Research is currently underway. In the present study the method of P-Delta (structure deformation) effect in multi-storied structures are identified floor wise and the significance of building responses like displacement, column moment, beam moment, column shear and beam shear are studied in detail. Seismic analysis is carried out as per IS-1893 (Part-I) 2002 guidelines. Equivalent static force method is adopted.

The stress resultants are displacement, bending moment and corresponding shear force. Linear elastic plane frame analysis is performed for the different models of the building using *STAAD* software. The frame members are modelled with rigid end zones.

A building has plan dimensions 20 m x 20 m with bay width 5 m both sides is selected. The building is located in Zone III as per IS 1893 (Part-I) – 2002)

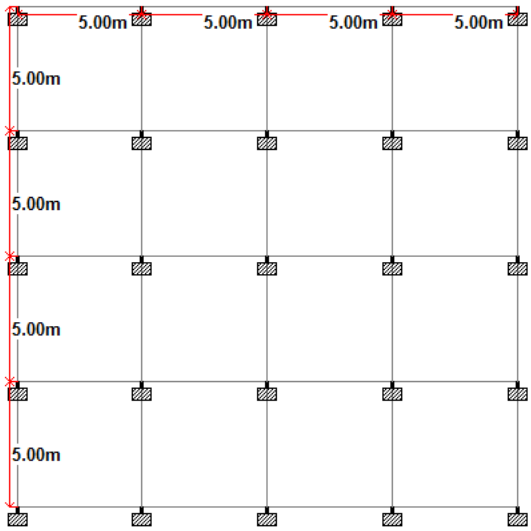

Fig. 1 Typical plan of Building

Particular	*RCC Structure*
Plan Dimension	20 x 20 m
Height of each storey	3.0 m
Size of Beam	0.3 x 0.45 m
Size of Column	0.45 x 0.45 m
Thickness of slab	0.125 m
Seismic zone	III
Earthquack load	As per IS:- 1893-2002
Type of soil	Type II medium as per IS:-1893-2002
Live load	2 kN/m2 at typical floor 1.5 kN/m2 at terrace.
Floor finish	1.25 kN/m2
Walls	0.23 m thick
Support at base	Fixed

TABLE III.1 Data of Example

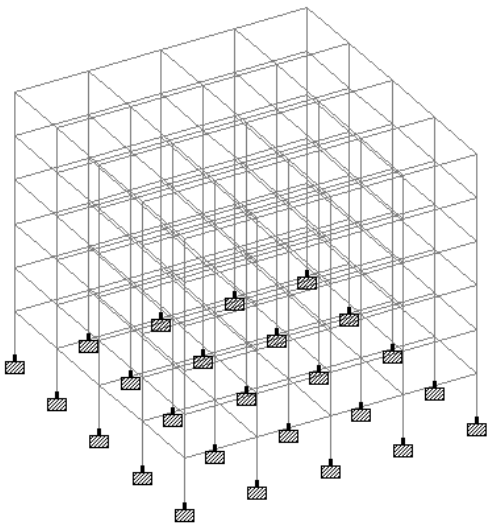

Fig. 2 3-Dimension view of Building

IV. MODELING AND ANALYSIS

Finite element models are developed to notice the variation of responses due to the effects obtained from static nonlinear analysis against linear static analysis using STAAD Pro v8i Software. In present study the building model has been analyzed for G+5 story. The maximum response in building Model had been studied. Lateral load for the selected frame has been carried out as per IS-1893 (Part-I) 2002. The analysis has been carried out for without P-Delta effect and then same has been analyzed for P-Delta effect with number of iterations. The maximum response values are compared to notify the P-Delta effect.

V. RESULTS

Response of building	*Without P-Delta*	*With P-Delta*
Maximum displacement	27.671 mm	27.675
Maximum beam moment	117.992 kN.m	117.996
Maximum column moment	186.034 kN.m	186.035
Maximum beam shear	136..267 kN	136.267
Maximum column shear	2298.374 kN	2298.375

TABLE V.1 Maximum Response of Building

VI. CONCLUSION

Present study is first stage of work were G+5 Story model has been analyzed and it is concluded that P-delta effects are negligible up to 5 storey buildings where only gravity loads are governing load combinations. As number of stories increases means height of building increases the P-Delta effect becomes more and more predominant. Thus in future work models having 10 to 30 story with 5 story interval will be carried out for further study.

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TECH-CHRONICLE
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EMERGING TRENDS IN SCIENCE, TECHNOLOGY AND MANAGEMENT

Earthquake Analysis Of ESR

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ABSTRACT- **Water supply is a life line facility that must remain functional following disaster. Most municipalities in India have water supply system which depends on elevated water tank for storage. Elevated water tank is a large elevated water storage container constructed for the purpose of holding a water supply at a height sufficient to pressurize a water distribution system. These tanks must remain functional in post earthquake period and toxic contents in them should not leak. In india, provisions for a seismic design of liquid storage tanks are given in IS 1893:1984. There are some limitations and shortcomings in the provision of IS 1893:1984 which has been revised in a new set of provisions for a sesmic design of tank. The focus of this study is to compare various quantities obtained by using the proposed modified provisions and existing practice of IS 1893:1984.for that some examples are included to illustrate application of proposed revision. A comparision is also made between the results obtain by applying the proposed revision and existing provision to the same design problem in order to get an idea of the implication of proposed revision. Some major modifications are given in proposed draft code these modified provisions suggested in this paper can be readily adopted for IS 1893 (PART 2).**

KEYWORDS: ***Overhead water tank, seismic analysis, convective mass and impulsive mass.***

1. INTRODUCTION

Indian sub continent is highly vulnerable to natural disasters like earthquakes, draughts, floods, cyclones etc. Majority of states or union territories are prone to one or multiple disasters. These natural calamities are causing many casualties and innumerable seismic code IS 1893 (PART 1): 2002, more than 60% of India is prone to earthquakes. The goals of seismic criteria for the design and construction of structures in present day standards are i) to minimize the hazard to life ii) increase the expected performance of structures with a greater importance or hazard to the public iii) to improve the capability of structures essential for the welfare of the public after an earthquake.

Water is considered as the source of every creation and is thus a very crucial element for humans to live a healthy life. High demand of Clean and safe drinking water is rising day by day as one cannot live without water. It becomes necessary to store water. Sufficient water distribution depends on design of a water tank in certain area. An elevated water tank is a large water storage container constructed for the purpose of holding water supply at certain height to pressurization the water distribution system. These structures have a configuration that is especially vulnerable to horizontal forces like earthquake due to the large total mass concentrated at the top of slender supporting structure. Seismic safety of liquid storage tank is of considerable importance. In india, provisions for a seismic design of liquid storage tanks are given in IS 1893:1984. These provisions are only for elevated tanks and there are no provisions for ground supported tanks. The focus of this study is to compare various quantities obtained by using the proposed modified provisions and existing practice of IS 1893:1984. O.R. Jaiswal , Durgesh C. Rai, M. EERI, and Sudhir K. Jain (2003) in this paper, revisions of ten seismic codes on tanks are reviewed and compared. This review has revealed that there are significant differences among these codes on design seismic forces for various types of tanks. Mohammad Taghi Ahmadi, Afshin Kalantari (2002) in this research, a simplified containing the first and second sloshing modes and the first structural mass is suggested. This study expresses that: in some conditions a tank which is not full has been observed with larger responses comparing with a full tank. In some conditions code results are less than the results, which are obtained in models, which consider the effect of sloshing modes. Praveen k. Malhotra, Thomas Wenk, Maartin Wieland (2000), this paper provides the theoretical background of a simplified seismic design procedure for cylindrical ground supported tanks. The procedure takes into account impulsive and convective(sloshing) action of the liquid in flexible steel or concrete tanks fixed and rigid foundations. Durgesh C. Rai and Bhumika Singh (2003), this paper will review the existing code design procedures in the light of actual performance data will suggest modifications for safer designs. This paper studies both tension – flexure mode and shear mode of failure of shaft supports.

2. CODAL PROVISION

The Criteria given in IS 1893: 1984 for the seismic analysis of elevated water tank. For the purpose of this analysis, elevated tanks shall be regarded as systems with a SDOF with their mass concentrated at their centers of gravity. The damping in the system may be assumed as 2 percent of the critical for steel structures and 5 percent of the critical for concrete structures. In modified provisions, dynamic analysis of liquid containing tank is a complex problem involving fluid – structure interaction.

Most elevated tanks are never completely filled with liquid. Hence a two-mass idealization of the tank is more appropriate as compared to a one-mass idealization, which was used in IS 1893: 1984. Two mass model for elevated tank was proposed by Housener, and is being commonly used in most of the international codes. Structural mass ms include mass of container and one-third mass of staging. Mass of container comprises of mass of roof slab, container wall, gallery, floor slab, and floor beams. Staging acts like a lateral spring and one-third mass of staging is considered based on classical result on effect of spring mass on natural frequency of single degree of freedom system. The response of the two-degree of freedom system can be obtained by elementary structural dynamics.

3. *DESIGN OF CIRCULAR WATER TANK*

A RC circular water container of 1000 m^3 capacity has internal diameter of 14 m and height of 8.0 m (including freeboard of 0.3 m). It is supported on RC staging consisting of 8 columns of 650 mm diameter with horizontal bracings of 300 X 450 mm. The lowest supply level is 15 m above ground level. Staging conforms to ductile detailing as per IS 13920. Staging columns have isolated rectangular footings at a depth of 3m from ground level. Tank is located on soft soil in seismic zone IV. Grade of staging concrete and steel are M25 and Fe415, respectively. Density of concrete is 25 kN/m^3. Analyse the tank for seismic loads.

Tank must be analysed for tank full and empty conditions.

Preliminary data:

Details of sizes of various components and geometry are

Capacity	**V**	**1000**	**m^3**
Density of water	**ρ**	**10**	**KN/m^3**
C/C Dia. Of Container	**D1**	**14.2**	**M**
Inner Dia. Of Container	**D**	**14**	**M**
Staging Height		**15**	**M**
No of steps	**N_S**	**3**	**Nos**
Roof slab		**0.12**	**M**
Roof slab diameter		**14.4**	**M**
Roof beam width		**0.25**	**M**
Roof beam depth		**0.52**	**M**
Wall Thickness		**0.2**	**M**
Depth of water		**7.7**	**M**
Av thickness of balcony		**0.13**	**M**
Base slab thickness		**0.2**	**M**
Base beam width		**0.3**	**M**
Base beam depth		**0.5**	**M**
Span of Bottom Beam		**4**	**M**
Types of soil		**Hard**	
Depth of footing		**3**	**M**
Column diameter		**0.45**	**M**
Brace width		**0.3**	**M**
Brace depth		**0.4**	**M**
Density of concrete	**ρ_c**	**25**	**KN/m^3**

Seismic Data		
Zone	**Z**	**0.36**
Importance factor	**I**	**1.5**
Response Reduction	**R**	**2.25**

4. *DESIGN OF ELEVATED SERVICE RESERVOIR*

A RC circular water container of 250 m^3 capacity has internal diameter of 8.6 m and height of 2.5 m (including freeboard of 0.3 m). It is supported on RC staging consisting of 6 columns of 650 mm diameter with horizontal bracings of 300 X 600 mm. The lowest supply level is 16.3 m above ground level. Staging conforms to ductile detailing as per IS 13920. Staging columns have isolated rectangular footings at a depth of 4 m from ground level. Tank is located on soft soil in seismic zone IV. Grade of staging concrete and steel are M20 and Fe415, respectively. Density of concrete is 25 kN/m^3. Analyse the tank for seismic loads.

Tank must be analysed for tank full and empty conditions.

Preliminary data:

Details of sizes of various components and geometry are

Capacity	**V**	**250**	**m^3**
Density of water	**P**	**10**	**KN/m^3**
C/C Dia. Of Container	**D1**	**8.8**	**M**
Inner Dia. Of Container	**D**	**8.6**	**M**
Staging Height		**16.3**	**M**
No of steps	**N_S**	**3**	**Nos**
Roof slab		**0.12**	**M**
Roof slab diameter		**6.28**	**M**
Roof beam width		**0.25**	**M**
Roof beam depth		**0.52**	**M**
Wall Thickness		**0.2**	**M**
Depth of water		**2.2**	**M**
Av thickness of balcony		**0.13**	**M**
Base slab thickness		**0.2**	**M**
Base beam width		**0.3**	**M**
Base beam depth		**0.5**	**M**
Span of Bottom Beam		**4**	**M**
Types of soil		**Hard**	
Depth of footing		**3**	**M**
Column diameter		**0.65**	**M**
Brace width		**0.3**	**M**
Brace depth		**0.6**	**M**
Density of concrete	**ρ_c**	**25**	**KN/m^3**

Seismic Data

Zone	**Z**	**0.36**
Importance factor	**I**	**1.5**
Response Reduction	**R**	**2.25**

5. *RESULTS*

COMPARISION OF RESULTS OBTAINED BY MODIFIED PROVISIONS AND IS 1893:1984		
Idealization of tank	**Two- Mass**	**One- Mass**
1. Lateral stiffness of staging	**3845.15 kN/m**	**286823.80kN/m**
2. Time period, Impulsive mode **Tank empty** **Tank full (Ti)** **Convective mode,** **Tank full (Tc)**	 **Ti = 3.51 sec** **Ti = 2.07 sec** **Tc = 4.04 sec**	 **Ti = 0.46 sec** **Ti = 0.94 sec**
3. Design seismic horizontal coefficient		

COMPARISION OF RESULTS OBTAINED BY MODIFIED PROVISIONS AND IS 1893:1984		
Idealization of tank	**Two- Mass**	**One- Mass**
1. Lateral stiffness of staging	**17806 kN/m**	**55089 kN/m**
2. Time period, Impulsive mode		
Tank empty	**Ti = 0.66 sec**	**Ti = 0.37 sec**
Tank full (Ti)	**Ti = 0.86 sec**	**Ti = 0.57 sec**
Convective mode,		
Tank full (Tc)	**Tc = 3.14 sec**	
3. Design seismic horizontal coefficient		
Impulsive mode		
Tank empty (Ah)i	**Ahi = 0.11 sec**	**Ahi = 0.075 sec**
Impulsive mode		
Tank empty (Ah)i	**Ahi = 0.04 sec**	**Ahi = 0.012 sec**
Tank full (Ah)i	**Ahi = 0.02 sec**	**Ahi = 0.007 sec**
Convective mode,		
Tank full (Ah)c	**Ahc = 0.03 sec**	
4. Base shear (V)		
Tank empty	**V = 145.24 KN**	**V = 45.20 KN**
Tank full	**V = 297.56 KN**	**V = 111.29 KN**
5. Overturning moment (M)		
Tank empty	**M = 2338.23 kN. m**	**M = 1224.0 kN. m**
Tank full	**M = 6531.43 kN. m**	**M = 2332.0 kN.m**

Tank full (Ah)i **Convective mode,** **Tank full (Ah)c**	**Ahi = 0.084 sec** **Ahc = 0.04 sec**	**Ahi = 0.056 sec**
4. Base shear (V) **Tank empty** **Tank full**	 **V = 212 KN** **V = 281 KN**	 **V = 144 KN** **V = 248 KN**
5. Overturning moment (M) **Tank empty** **Tank full**	 **M = 4053 kN. m** **M = 5448 kN. m**	 **M = 2762 kN. m** **M = 4757 kN.m**

6. *CONCLUSION:-*

In this dissertation detailed commentary describing the rationale of modified provision is provided. Solved examples are also included to illustrate the application of these modified provision.

In modified provisions, values of response reduction factor for different types of tanks have been provided. Illustrative

solved example has clearly shown that for some ground supported tanks, design will be influenced by hydrodynamic forces. This clearly brings out the need for seismic analysis of ground supported tanks, which was not considered in IS 1893:1984. From the sloved example on elevated tank it is seen that flexibility of brace beams, which was not considered in, IS 1893:1984. It is seen that provisions of IS 1893:1984 can grossly underestimate design forces for certain elevated tanks. The major modifications are: i) Design horizontal seismic coefficient as given in IS 1893 (part 1): 2002 has been used for tanks and suitable values of importance factor I and response reduction factor R are proposed.

ii) Spring mass model of for tanks with rigid and flexible wall has been included.

iii) Some errors in the expression for convective hydrodynamic pressure are rectified.

The provisions suggested in this paper can be readily adopted for IS 1893 (Part 2)

Earthquake forces decreases with increase in staging height and increases with increase in zone for tank empty and full condition. Earthquake forces increases from zone II to zone V for tank empty and tank full condition. Tank full condition is more severe as compared to tank empty conditions. Earthquake forces decreases with increase in staging height because as staging height increases, the structure becomes more flexible. Therefore time period increase due to which structural response factor decreases from lower to higher staging height. This affects the earthquake forces.

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TECH-CHRONICLE
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" Seismic Analysis Of Vertically Irregular RC Building"

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ABSTRACT

From past earthquakes it is proved that many of structure are totally or partially damaged due to earthquake. So, it is necessary to determine seismic responses of such buildings. There are different techniques of seismic analysis of structure. Time history analysis is one of the important techniques for structural seismic analysis generally the evaluated structural response is non-linear in nature. For such type of analysis, a representative earthquake time history is required. In this project work seismic analysis of RCC buildings with mass irregularity at different floor level are carried out. Here for analysis different time histories have been used. This paper highlights the effect of mass irregularity on different floor in RCC buildings with time history and analysis is done by using SAP 2000 software.

KEYWORDS: Seismic Analysis, Time History Analysis, Irregular Structure , Story Drift

I. INTRODUCTION

Earthquakes have the potential for causing the greatest damages, among all the natural hazards. Since earthquake forces are random in nature & unpredictable. During an earthquake, the damage in a structure generally initiates at location of the structural weakness present in the building systems.These weaknesses trigger further structural deterioration which leads to the structural collapse which is due to geometry, mass discontinuity and stiffness of structure. The structures having this discontinuity are known as **Irregular structures**. These structures constitute a large portion of the modern urban infrastructure. Vertical irregularities are one of the major reasons of failures of structures during earthquakes. For example structures with soft storey were the most notable structures which collapsed. So, the effect of vertically irregularities in the seismic performance of structures becomes really important. Height-wise changes in stiffness and mass render the dynamic characteristics of these buildings different from the regular building. As per IS 1893, the irregularity in the building structures may be due to irregular distributions in

their mass, strength and stiffness along the height of building. When such buildings are constructed in high seismic zones, the analysis and design becomes more complicated hence structural engineer needs to have a thorough understanding of the seismic response of irregular structures.

1.1 Types of Irregularities

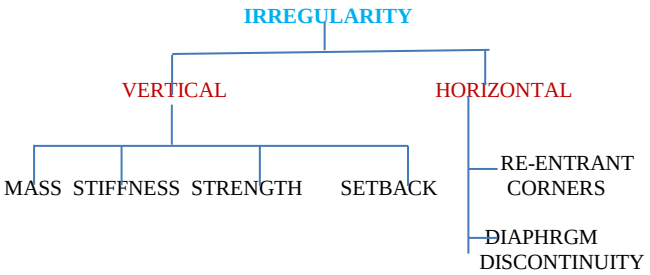


Fig 1. Types Of Irregularities

In recent past, several studies have been carried out to evaluate the response of irregular buildings. Soni A.(2015) discussesed the performance evaluation of RC Buildings with various frames with different irregularities & proves that irregularities in buildings are harmful for the structures and it is important to have simpler and regular shapes of frames as well as uniform load distribution of load around the building. S. Mahesh & Rao B.P. (Nov 2014) studied the behavior of multistoried building of regular & irregular configuration assuming earthquake. For problem formulation residential G+11 multi story building is used.Assuming that material property is linear static and dynamic & by considering different seismic zones and three different types of soils namely Hard , Medium and Soft .The analysis were done by two softwares ETABS and STAAD PRO & proves STAAD PRO has more values.By using the

soft computing tool and commercial software CSI-ETABS , Shaikh Abdul & Deshmukh G. (2014) studied the response of a G+ 10-storeyed vertically irregular frame to lateral loads for stiffness irregularity .The seismic analysis of RCC building with mass irregularity at different floor levels have been studied by Padol S. & Talikoti R.(2015). Poonam & Anilkumar (2012) investigated the response of a 10-storeyed plane frame to lateral loads for mass and stiffness irregularities in the elevation introduced by changing the properties of the members of the storey under consideration. Soni D. (2006) agreed on the increase in drift demand in the lower portion of set-back structures and on the increase in seismic demand for buildings with discontinuous distributions in mass, strength and stiffness. The largest seismic demand was found for the combined stiffness and strength irregularity. Sadjadi (2007) presented an analytical approach for seismic assessment of RC frames using nonlinear time history analysis and push-over analysis & concluded that both the ductile and the less ductile frames behaved very well under the earthquake considered, while the seismic performance of the GLD structure was not satisfactory. Athanassiadou C.J. (2008) concluded that the effect of the ductility class on the cost of buildings is negligible, while performance of all irregular frames subjected to earthquake appears to be equally satisfactory, not inferior to that of the regular ones, even for twice the design earthquake forces. Das (2003) evaluated the effects of stiffness, strength and mass irregularity on inelastic seismic response of large number of multistorey structures using two different codes namely ACI 1999 and UBC 97 it was concluded that the seismic response parameters like first mode shape and fundamental time period as computed by ELF procedure were similar for symmetrical and unsymmetrical structure & further extended the work Das (2000) found that most of the structures designed by ELF method performed reasonably well. Capacity based criteria must be appropriately applied in the vicinity of the irregularity. Modakwar N. & Sangita M. (2015) studied the different irregularity and torsional response due to plan and vertical irregularity and to analyze cross shape and L shape building while earthquake forces acts and to calculate additional shear due to torsion in the columns. Semih S. & Cenk A. focused on the performance on nonorthogonal and torsional irregular structures and investigate the effects of torsion on moment and shear values of vertical structural elements. The practical considerations and vibration control effectiveness of passive tuned mass dampers (PTMDs) for irregular buildings illustrates by Chi-Chang Lin , Jin-Min Ueng, Teng-Ching Huang Developed a methodology for the derivation of fragility relationships for three-dimensional (3D) structures with plan irregularities by Seong-Hoon Jeong , Amr S. Elnashai. Habibi A. & Asadi K. (2014) investigate RCMRFs regular and irregular in elevation with different types of setbacks, as well as the regular frames in elevation, are designed according to the Iranian code & shows that when setback occurs in elevation, the requirements of the life safety level are not satisfied , therefore it is necessary to strengthen these elements by appropriate method to satisfy the life safety level of the frames.In this work, the main aim is to investigate the effects of various vertical irregularities on the seismic response of a structure in various seismic zones of India.The objective is to carry out Response spectrum analysis (RSA) of vertically irregular building. Mainly the Two types of irregularities i.e mass irregularity, stiffness irregularity were considered.The building frame is considered with vertical irregularity & have been analyzed using Response Spectrum method of IS 1893- Seismic Response of Vertically Irregular RC Frame with Stiffness Irregularity 341 part 1: 2002 .

II. BUILDING DESCRIPTION

In the earlier versions of IS 1893 (BIS, 1962, 1966, 1970, 1975, 1984), there was no provision of vertical irregularity in building frames. However, in the recent version of IS 1893 (Part 1)-2002 (BIS,2002), irregular configuration of buildings has been defined explicitly hence the problem considered for the current study is taken in reference to IS 1893-part 1:2002.This G+10 building frame is considered as a regular structure have been analyzed using Time History Method. There are some assumptions used for building are as follows

Frame-1: This is the regular plan of the building with no irregularities and 11 storeys.

Frame-2: This is the irregular plan of the building with mass irregularity in the of swimming pool at the 4th floor

Frame-3: This frame carries heavier loading on the 8th floor i.e. swimming pool has been introduced hence making building mass irregular.

Frame-4: Frame with swimming pool at its 10th floor making building irregular.

The data assumed for the problem to be analysing in SAP 2000 are as follows:

Table 1 Section Properties

Columns Designation	Size (mm)	Beams Designation	Size (mm)
C1	350 X 700	B1	230 X 300
C2	350 X 700	B2	230 X 350
C3	350 X 700		

- Building = (G + 10) storey
- Slab thickness = 120 mm
- Live Load on floor = 3 KN/m^2
- Live Load on terrace = 1.5 KN/m^2
- Storey Height = 3 m
- Software Used = SAP 2000 v14.2.4

- Method of Analysis = Nonlinear Time History
- Earthquake used = 1) Loma Prieta 2) Sanfransisco
  3) Nridge 4) Park 40

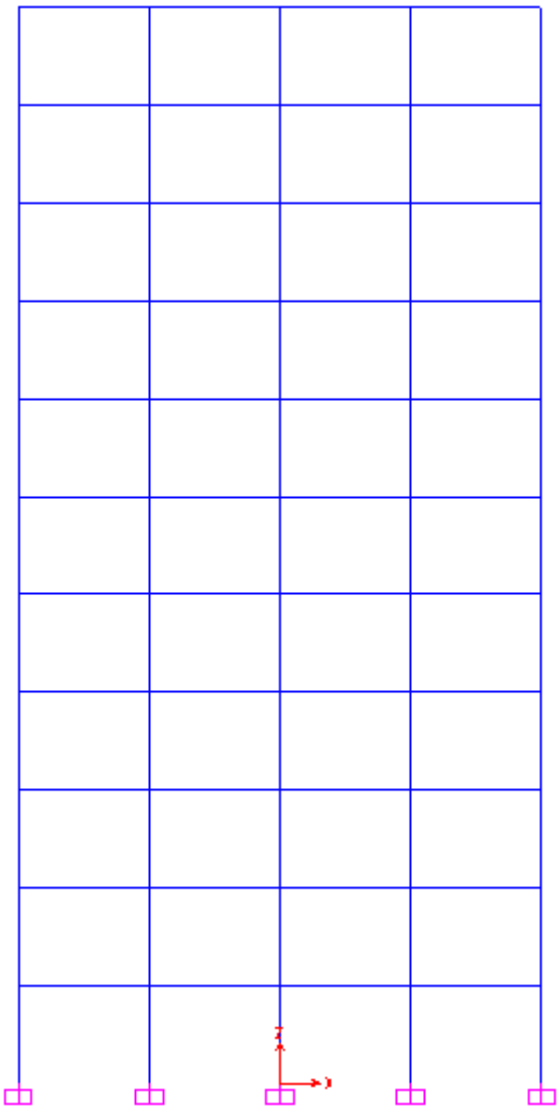
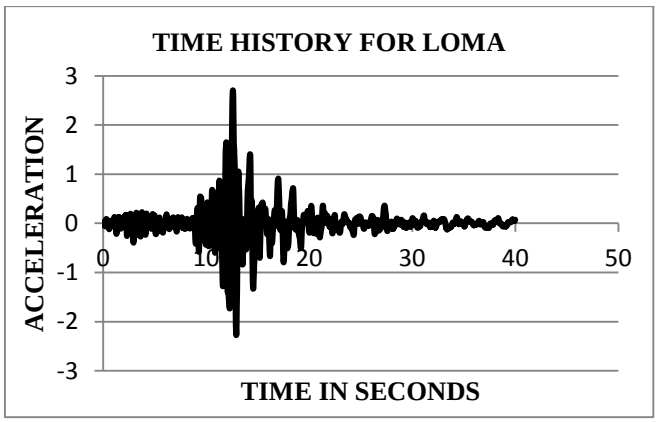
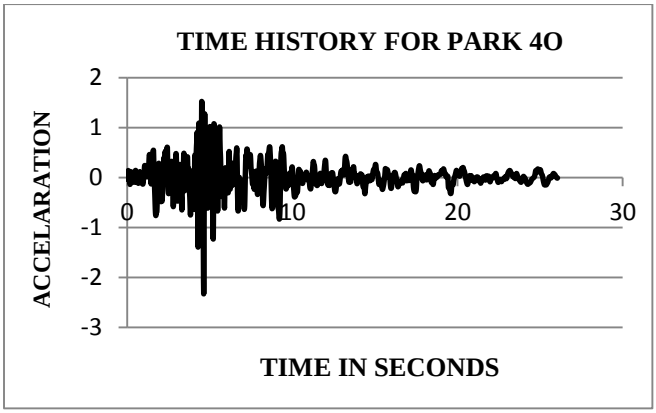
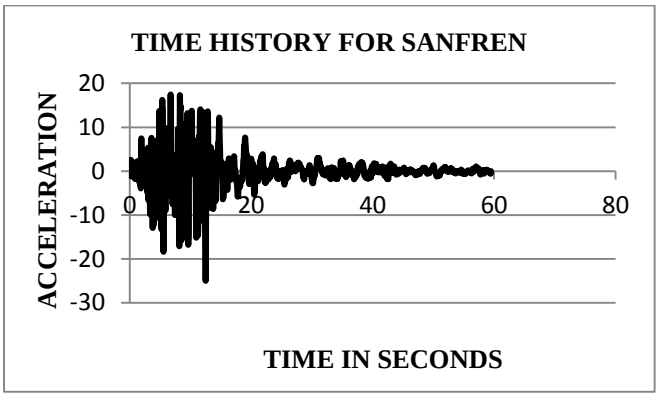


Fig 2. Elevation of building







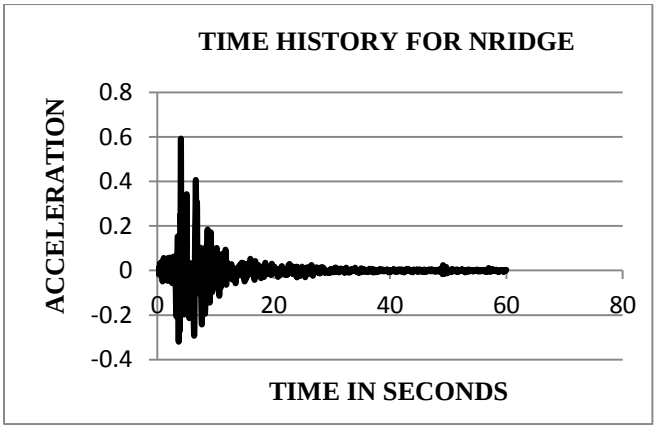


Fig 3. Real Ground Motion Records

III. ANALYSIS RESULTS

The four frames were analysed using the time history & their displacement & storey drifts results for various building frames have been computed.The results are presented in fig.4

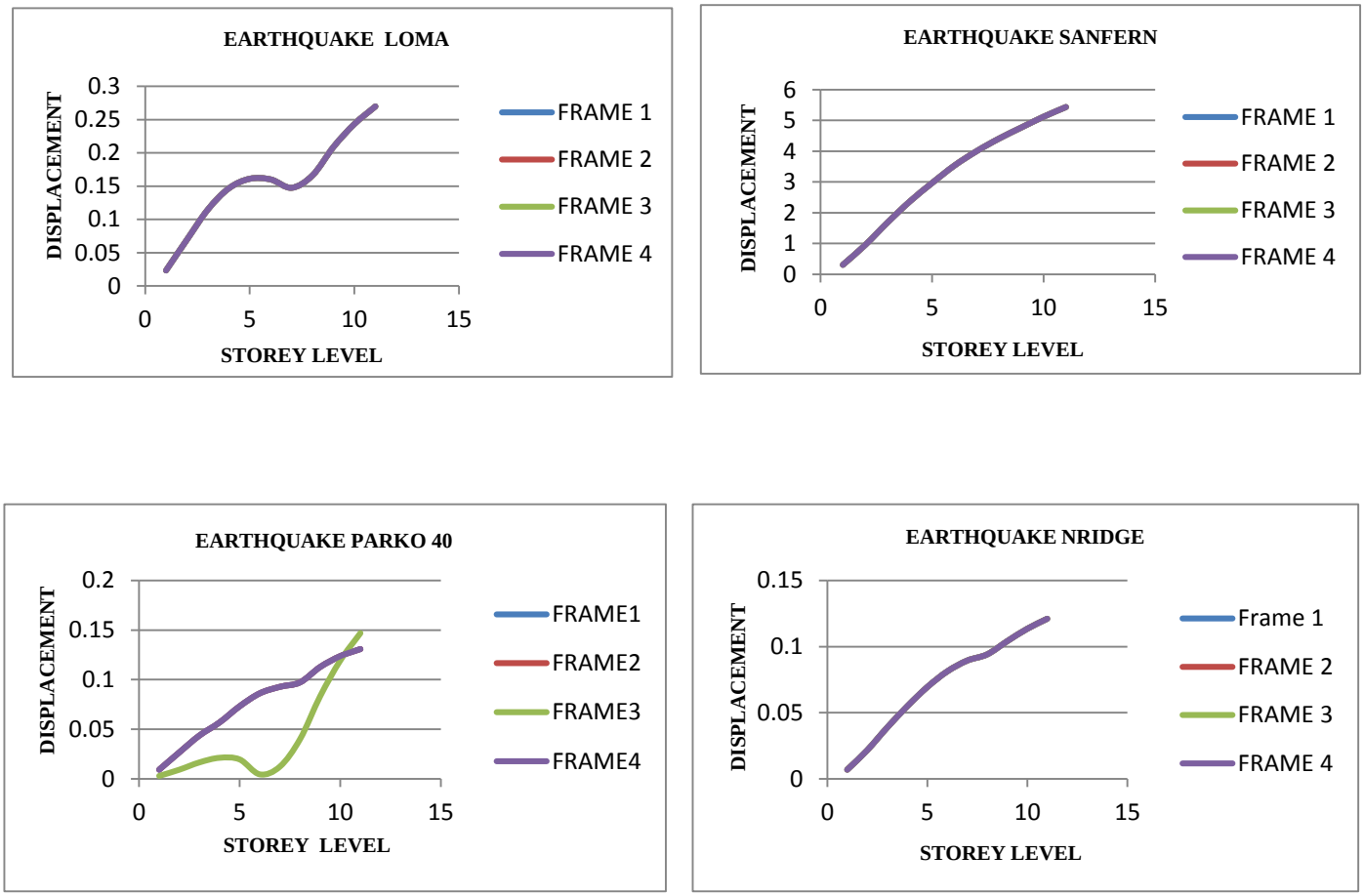


Fig 4. Storey Displacement Graphs

Frame 1- Regular building frame

Frame 2- Mass irregular frame with swimming pool at 4th floor

Frame 3- Mass irregular frame with swimming pool at 8th floor

Frame 4- Mass irregular frame with swimming pool at 10th floor

From the above graphs of storey displacement it has been clear that only frame 3 shows the change in response due to irregularity i.e means there may be chances of change in response due to irregularities in the structure.

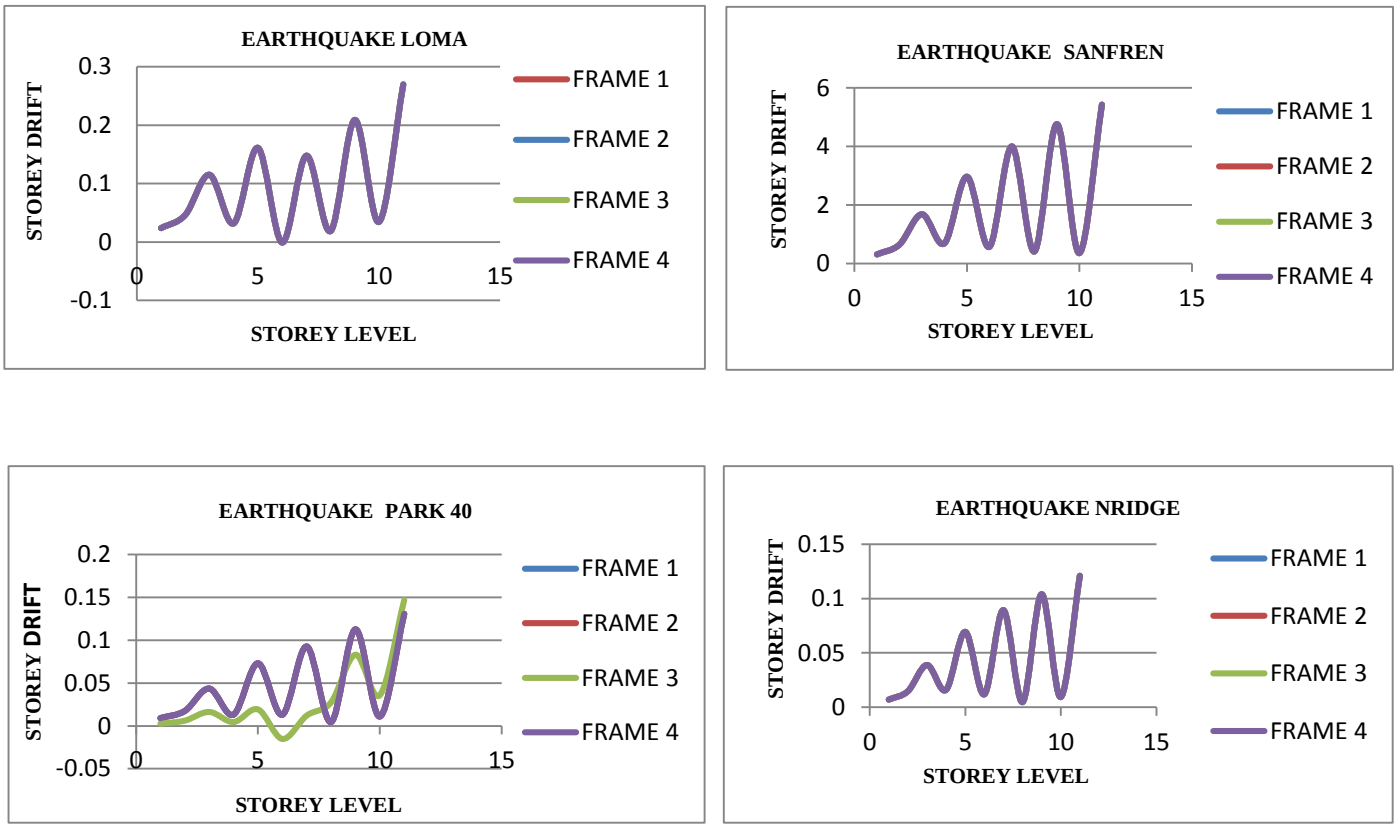


Fig 5. Storey Drift Graphs

From the above graphs of storey drift it has been clear that all frames are seen to exhibit abrupt changes in storey drifts, which is highly undesirable.i.e means there may be chances of change in response due to irregularities in the structure.

IV. CONCLUSION

In dynamic nonlinear analysis by using four real ground motion records Loma prieta , Park 40, Nridge & Sanfern shows nonlinear behaviour of structure with & without irregularity.The Displacement & Storey Drifts of all the structures are compared.There may be chances of change in response due to irregularities in the structure therefore out of 3-irrregular frames 1frame has shown different results as compared to other i.e frame-3. Same behaviour observed in the drift response also.

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TECH-CHRONICLE
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PERFORMACE BASED DESIGN OF OMRF & SMRF FRAME

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Abstract-

This paper investigates seismic performance and vulnerability analysis of 4-storey and 6-storey code-conforming (IS: 456-2000, Indian standard for plain and reinforced concrete code and IS: 1893-2002, Indian standard criteria for earthquake resistant design of structures) reinforced concrete (RC) buildings. The buildings are designed for two different cases such as ordinary moment resisting frame (OMRF) and special moment resisting frame (SMRF). The nonlinear static analysis (pushover analysis) is used to capture initial yielding and gradual progressive plastic behavior of elements and overall building response under seismic excitations. The deformation characteristics of structural elements are essential to simulate the plastic hinge formation in the process of generation of capacity curve during the pushover analysis. An analytical procedure is developed to evaluate the yield, plastic and ultimate rotation capacities of beams and columns along with different plastic hinge lengths. In the present study, user defined plastic hinge properties of beams and columns are modeled using analytical expressions developed based on Eurocode 8 and incorporated the same in pushover analysis using SAP2000.

***Keywords*: Plastic hinge length, Seismic performance, nonlinear static analysis, Pushover analysis, SMRF, OMRF etc.**

I. INTRODUCTION

The nonlinear static analysis, to evaluate the seismic performance of buildings, represents the current trend in structural engineering and promises a reasonable prediction of structural behavior. The analysis provides adequate information on seismic demands imposed by the design ground motion on the structural system and its components. Earthquake is a phenomenon related to violent shaking that takes place underneath the earth. Massive strain energy discharged at the time of an earthquake and travels as unstable waves called as seismic waves in every directions through the Earth's layers, which refracting and reflecting at every interface. The destruction to structures because of earthquake depends on the stuff that the structure is formed out of, the sort of earthquake wave (motion) that is distressing the structure, and also the ground on that the structure is constructed. Therefore the dynamic loading which acts on the structure throughout an earthquake is not only external loading, but also inertial effect caused by motion of support. The different factor that causes damage to the structure throughout earthquake is mass irregularity, vertical irregularities, tensional irregularity, irregularity in strength and stiffness, etc. In multi-storied RC framed buildings, destruction from earthquake ground motion usually starts at locations of structural weaknesses there in buildings. In some of the cases, these weaknesses are also developed by discontinuities in stiffness, strength or mass between adjacent stories.

Over the past decades it has been recognized that destruction control has become a more specific design consideration which will also be carried out most effectively, by the way of introducing some kind of nonlinear analysis into the seismic design methodology. Following this pushover analysis has been developed during past years and has end up with the preferred method of analysis for performance-based seismic design (PBSD). It is the approach by which the ultimate strength and the limit state can be quite simply investigated after yielding, which has been researched and utilized in practice for earthquake engineering and seismic design.

II. BUILDING PERFORMANCE LEVEL

Building Performance Level A performance level describes a limiting damage condition which may be considered satisfactory for a given building and a given ground motion. The limiting condition is described by the physical damage within the building, the threat to life safety of the building's

occupants created by the damage, and the post-earthquake serviceability of the building.

ATC-40 describes standard performance levels for structural performance as:

a. Operational

b. Immediate Occupancy (IO): very limited structural damage has occurred. The risk of life-threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.

c. Damage Control: a range of IO and Life Safety (LS). It limits the structural damage beyond the Life Safety level, but occupancy is not the issue. E.g. the protection of significant architectural features of historic buildings or valuable contents

d. Life Safety (LS): the injuries during the earthquake may occur; the risk of life-threatening injury from structural damage is very low.

e. Limited Safety: a range of LS and Collapse Prevention (CP), Structural Stability. Some critical structural deficiencies are mitigated.

f. Structural Stability or Collapse Prevention (CP): Substantial damage to the structure has occurred, including stiffness and strength of the lateral force resisting system. However, all significant components of the gravity load resisting system continue to carry their gravity demands.

g. Not considered: The performance level of a building is determined based upon its function and importance. Public building is expected to have a performance level of operational or immediate occupancy. A residential building must have a performance level of damage control or life safety. For the temporary structure is 16 under the structural stability or not considered. The force deformation relationship as well as the structural performance levels is given in Figure 2.3.

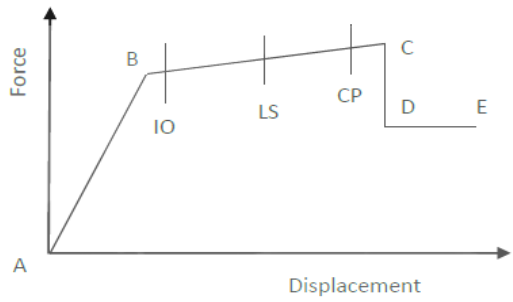

Fig 1: Force-deformation curve

Where:

A = the origin

B = yielding

IO = immediate occupancy

LS = life safety

CP = collapse prevention

C = ultimate capacity

D = residual strength

E = total failure

Five points labeled A, B, C, D, and E are used to define the force deflection behavior of the hinge. Three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge.

III. OBJECTIVE OF THE WORK

The main objectives of this work includes the following,

1) To determine the response of 7 storey RC frame structure i.e., base shear and lateral displacement by Equivalent static lateral force method and performance point by pushover analysis. Modeling and analysis are achieved using SAP 2000

2) Equivalent static lateral force method is conducted for zone-II and zone-V according to IS 1893 2002 (Part 1) for soft soil type (type III).

3) All the models are studied and analyzed using pushover analysis.

IV. PARAMETRIC STUDY

A reinforced concrete frame with 7(G+6) storey of dimension 25mx12m, has been taken for seismic analysis.

a) Using equivalent static lateral force method for zone-IV for soil type-III (soft soil) as per IS 1893(part 1):2002.

b) Using Pushover analysis.

V. METHODOFANALYSIS

The study undertakes the following analysis

- Equivalent Static Lateral Force Method (ESLM).
- Pushover Analysis.

VI. DESCRIPTIONS OF BUILDING

Description of building	
Structure type	Ordinary Moment Resisting Frame[OMRF] & Special Moment Resisting Frame[SMRF]
Plan dimension	12x25m
Storey height	3m
Height of building	G+6=7 storeys
Grade of concrete	M25(beams and slabs) & M25 (columns)
Grade of steel	Fe415
Beam sizes	B1- 230 mm X 350 mm, B2- 230 mm X 300 mm

column sizes	C1-230 mm X 400 mm C2- 400 mm X 400 mm
slab thickness	150mm
Live load	2.0kN/m2
Floor finish	1.0kN/m2
Zone factor	IV
Soil type	Soft soil
Importance factor	1
Response reduction factor	3.0(OMRF) and 5.0(SMRF)

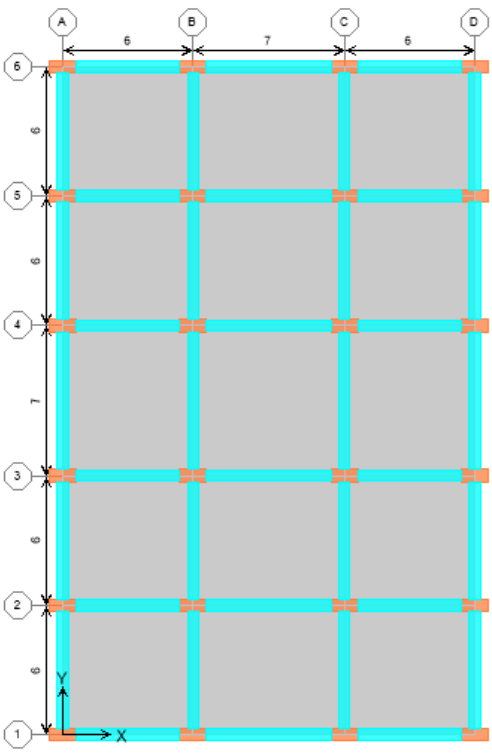

Fig 2: plan of building

VII. NONLINEAR STATIC PUSHOVER ANALYSIS

Analysis methods are broadly classified as linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis. In these the first two is suitable only when the structural loads are small and at no point the load will reach to collapse load. During earthquake loads the structural loading will reach to collapse load and the material stresses will be above yield stresses. So in this case material nonlinearity and geometrical nonlinearity should be incorporated into the analysis to get better result.

Nonlinear static pushover analysis or Push-over analysis is a technique by which a computer model of the building is subjected to a lateral load of a certain shape (i.e., parabolic, triangular or uniform). The intensity of the lateral load is slowly increased and the sequence of cracks, yielding, plastic hinge formations, and failure of various structural components is recorded. In the structural design process a series of iterations are usually required during which, the structural deficiencies observed in iteration is rectified and followed by another. This iterative analysis and design procedure continues until the design satisfies pre-established performance criteria. In the other hand, static pushover analysis evaluates the real strength of the structure so that it will be useful and effective for performance based design.

This method is considered as a step forward from the use of linear analysis, because they are based on a more accurate estimate of the distributed yielding within a structure, rather than an assumed, uniform ductility. The generation of the pushover curve also provides the nonlinear behavior of a structure under lateral load. However, it is important to remember that pushover methods have no rigorous theoretical basis, and may be inaccurate if the assumed load distribution is incorrect.

For example, the use of a load pattern based on the fundamental mode shape may be inaccurate if higher modes are significant, and the use of any fixed load pattern may be unrealistic if yielding is not uniformly distributed, so that the stiffness profile changes as the structural yields.

This analysis provides data on the strength and ductility of the structure which otherwise cannot be predicted. Base shear *versus* top displacement curve of the structure, called pushover curves, are essential outcomes of pushover analysis. These curves are useful in ascertaining whether a structure is capable of sustaining certain level of seismic load.
The basic steps of POA are:

1. Assume the nonlinear force-displacement relationship of individual elements of structure (including yield strength, post yield stiffness and stiffness degradation, etc)

2. Calculate the target displacement of structure

3. Select a reasonable lateral load pattern, and pushing the structure under this load pattern which is monotonically increasing step by step, when a structural member yields, then its stiffness is modified, until the roof displacement of structure is up to the target displacement or the structure collapses. At this time, the evaluation of seismic performance of structure is obtained.

The main output of a pushover analysis is in term of response demand versus capacity as shown in Figure Demand and capacity is mutually dependent. As displacements increase, the period of the structure lengthens. This is reflected directly in the capacity spectrum. Inelastic displacements increase damping and reduce demand. The capacity spectrum method reduces demand to find an intersection with the capacity spectrum where the displacement is consistent with the implied damping. The intersection between capacity and demand curve develop the

performance point. At the performance point, capacity and demand are equal. The displacement of the performance point is the target displacement.

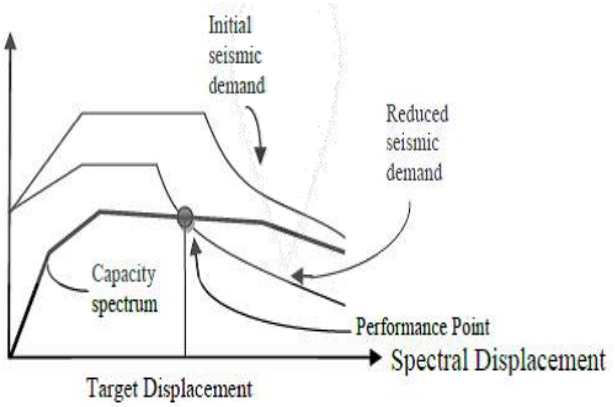

Fig 3: Demand vs Capacity Curve

Target Displacement and Lateral Load Pattern

The target displacement and lateral load pattern is very important for POA to evaluate the seismic performance of structures. The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The load patterns are intended to represent and bound the distribution of inertia forces in a design earthquake.

One procedure for evaluating the target displacement as per FEMA 356 is given by the following equation:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g$$

VIII. RESULTS & DISCUSSIONS

BEHAVIOR OF OMRF & SMRF STRUCTURAL SYSTEM

The behavior of OMRF & SMRF is taken as a basic study on the structures. The later forces resisting system is done for each building categorized based on lateral loads, lateral drifts, orientation of the shear wall & material quantity in terms of steel reinforcement alone. The modeled frame is a multi storied structure with a 12 m x 25 m (rectangular plan) and area of 300 sqm which have a bay of 3m x 6 m.Lateral forces considered in seismic area Lateral drift/deflections are checked against the requirements of clause 7.11.1 of IS-1893-2002 i.e. under transient seismic load. Deflections are discussed below for the OMRF &SMRF structural system.

PLASTIC HINGE LENGTH

Plastic hinges form at the maximum moment regions of RC members. The accurate assessment of plastic hinge length is important in relating the structural level response to member level response. The length of plastic hinge depends on many factors. The following is a list of important factors that influence the length of a plastic hinge 1) level of axial load 2) moment gradient 3) level of shear stress in the plastic hinge region 4) mechanical properties of longitudinal and transverse reinforcement 5) concrete strength and 6) level of confinement and its effectiveness in the potential hinge region.

The roof displacement obtained in this study obviously show that the demands of 4-storey buildings are higher than those of 6-storey ones. Therefore, it is difficult to precisely estimate which building group is more vulnerable during a seismic event. However SMRF building shows higher capacity compared to OMRF. The study also reveals that the amount of transverse reinforcement plays an important role in seismic performance of buildings, as the amount of transverse reinforcement increases the sustained damage decreases. A profound variation in capacity and displacement are brought out by varying the plastic hinge length and designing the building as OMRF and SMRF. Table 1 shows the inelastic response displacements of the frame. It is observed that inelastic displacement of all the frames are within collapse prevention.

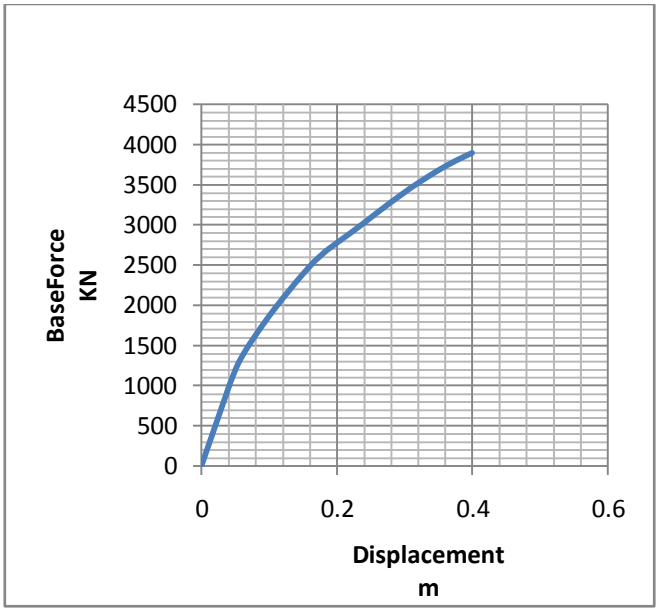

Fig.4 Capacity curves of six storey -OMRF

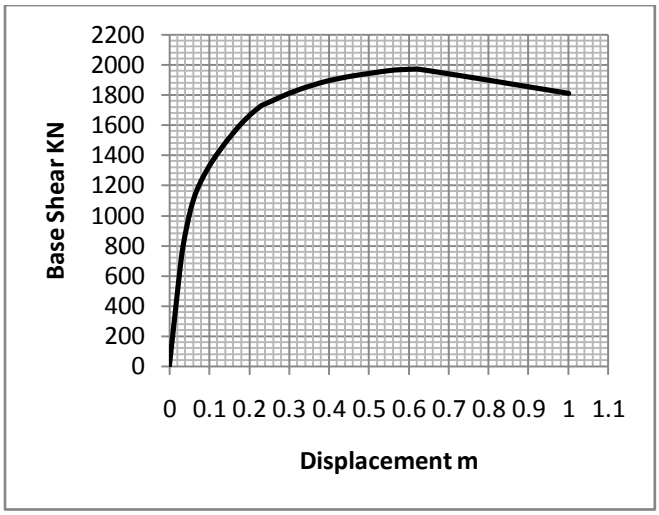

Fig.4 Capacity curves of six storey -SMRF

OMRF		SMRF	
Item	**Value**	**Item**	**Value**
C0	1.2579	C0	1.241
C1	1	C1	1
C2	1	C2	1
Sa	0.347	Sa	0.3856
Te	1.8444	Te	1.6598
Ti	1.8444	Ti	1.6598
Ki	23652.92	Ki	29785.49
Ke	23652.92	Ke	29785.49
Alpha	0.2677	Alpha	0.0643
R	5.8646	R	8.8376
Vy	1958.897	Vy	1444.515
Dy	0.0828	Dy	0.0485
Weight	33107.07	Weight	33107.07
Cm	1	Cm	1

Table:1

IX. CONCLUSIONS

This study has illustrated the nonlinear static analysis responses of OMRF and SMRF building frames under designed ground motions. The capacity against demand is observed significantly higher for SMRF building frames compared to OMRF. The user defined hinge definition and development methodology is also described. The user-defined hinges takes into account the orientation and axial load level of the columns compared to the default hinge. The influence of plastic hinge on capacity curve is brought out by deploying five cases of plastic hinge length. The study reveals that plastic hinge length has considerable effects on the displacement capacity of frames. Based on the analysis results it is observed that inelastic displacement of the modern code-conforming building frames are within collapse prevention level. The vulnerability index which is a measure of damage is estimated for both SMRF and OMRF are presented for 6-storey buildings. From the study it is apparent that, the OMRF framed buildings are more vulnerable than SMRF. The vulnerability index of the building quantitatively express the vulnerability of the building as such, where as storey vulnerability index assist to locate the columns in the particular storey in which significant, slight or moderate level of damages have taken place.

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TECH-CHRONICLE
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Study of shear connectors

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***Abstract*— This paper deals with study on the types, use and functions of shear connectors. The use of composite steel concrete material is increasing day by day and hence there is need of a material which can create a good bond between concrete and structural steel. The shear connector is one such material which satisfies all need and acts like a bridge in between structural steel and concrete material. These shear connectors are of different types and can be used as per the requirement, depending on the type of the structural members it has to connect.**

Keywords—composite material, structural steel, concrete

I. INTRODUCTION

The use of composite steel concrete material is increasing worldwide. As this composite material provide good strength and are economic and time effective and are the best solutions to many major civil structures mainly for bridges. Hence it requires one such material which can develop good bond between structural steel and concrete. The shear connector is such a material which satisfies all requirements and helps to form the good quality composite material.

The shear connectors are of different types depending on their type of the structure in which it has to be used. The shear connectors are designed as per the IRC code Standard Specifications and code of practice for Road Bridges, Section VI Composite Construction by Limit State Design (IRC-22-2015). The shear connectors essentially integrate the compression capacity of supported concrete slab to improve the load carrying capacity as well as over all rigidity of the member .

Shear connectors between concrete slabs and steel beams in composite construction can play an important role in the seismic response of a structure. They provide the necessary shear connection for composite action in flexure, and can be used to distribute the large horizontal inertial forces in the slab to the main lateral load resisting elements of the structure (Figure 1).

During an earth-quake, such shear connectors are subjected to reverse cyclic loading (Hawkins and Mitchell, 1984). This component enables the development of a composite action by assuring the shear transfer between the steel profile and the concrete deck (Vianna et al., 2009).

Connecting the existing concrete slab and steel girders is a potentially economic way to strengthen these floor systems as it allows for composite action to be developed.

In non-composite girders, the steel girders and the concrete slab act separately in flexure. Hence, by using shear connectors to connect the two structural components, the load-carrying capacity of the girders could be increased by more than 50% as compared to that of non-composite girders.

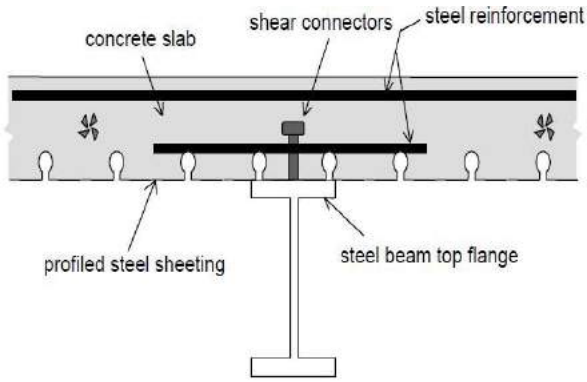

Fig 1. Shear connectors between concrete slabs and steel beams in a composite beam

Prior to casting the concrete slab, shear connectors are welded to the top of the steel girder in order to develop composite action in the construction of new bridges.

The design of shear connectors is a vital aspect in the design of composite beams. Shear connectors are of many types, and according to the distribution of shear forces and functional dependency between strength and deformation, they are often categorized as rigid or flexible. In this paper, an attempt has been made to review the different types of shear connector that can be found in composite structures.

II. TYPES OF CONNECTORS

i. HEADED STUDS

It resist horizontal shear and vertical uplift forces in composite steel-concrete structures, the most commonly used type of shear connector is the head stud.

This type of connector contributes to the shear transfer and

prevents uplift, as it is designed to work as an arc welding electrode, and, simultaneously, after the welding, acts as the resisting connector with a suitable head. As a result of the high degree of automation in the workshop or on site, this type of connector is commonly used worldwide. Much research has been carried out on headed stud shear connectors and various equations have been proposed to estimate the strength of studs (Viest, 1956a). Viest carried out the initial studies on stud shear connectors, where full-scale push out specimens were tested with various sizes and spacing of the studs. The push-out and composite beam tests were used in studies on stud shear connectors to evaluate shear capacities. In order to investigate the behavior of headed shear stud connectors in solid slabs, an accurate nonlinear finite element model were developed by Ellobody (2002) and Lam and Ellobody (2005).

Fig 2: Head stud shear connector

ii. WAVEFORM STRIPS

The objective of the curved form is to improve the transfer of force between the steel and the surrounding concrete as opposed to a straight connector. It is however recognized that it would be more difficult to weld using conventional automated welding equipment.

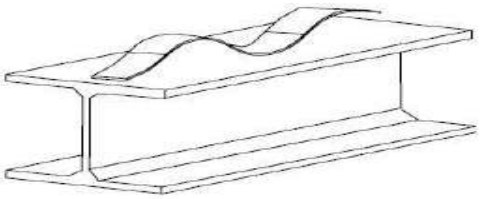

Fig 5.Waveform-strip shear connector

iii. T-CONNECTORS

This connector is a section of a standard T-section welded to the H or I section with two fillet welds. Therefore, a T section, which has a larger cross section than a single strip, and by its shape could prevent vertical separation between the steel-section and the concrete, seemed a good alternative. The behavior of the T-connector is very favourable. The beating stress on the front of the T is very high, as a result of the relatively small area. Local concrete crushing occurs, which results in a quasi-plastic performance. The load capacity for T-connectors is similar to that of the oscillating per bond strip, however, the ductility of these connectors is much larger (Rodera, 2008). When used in concrete with fibers, lightweight concrete or a higher strength concrete, there is a notable increase in the load capacity and ductility of this type of connector.

iv. CHANNEL CONNECTOR

Channel connectors might not need inspection procedures, such as bending test of headed studs, due to the highly reliable conventional welding system used in the welding of these connectors. The load carrying capacity of a channel shear connector is higher than that of a stud shear connector. This enables replacement of a large number of headed studs with a few channel connectors. Viest et al. (1952) reported on the test results of full size and push-out specimens. test was carried out on push-out specimens made of plain concrete, reinforced concrete (RC), fiber reinforced concrete (FRC) and engineered cementitious composite (ECC). Based on the results, the reversed cyclic shear strength of most specimens is lower than their monotonic strength by about 10 to 23%. The results also indicated that the shear strength and load-displacement behavior of the specimens is slightly affected by the use the polypropylene fibres (FRC specimens).

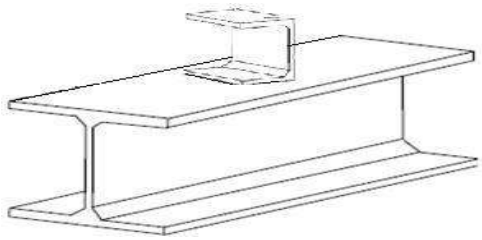

Fig 6.Channel shear connector

V. PYRAMIDAL SHEAR CONNECTORS

Sufficient bending strength and flexural rigidity for loads during and after construction is expected from a steel plate-concrete composite slab with pyramidal shear connectors (Figure 7). A TSC composite slab, which is composed of a bottom steel deck and concrete through pyramidal shear connectors, could also be one of them. The fatigue problem should play a significant role in design when such a TSC composite slab is applied to a bridge deck subjected to traffic loads.

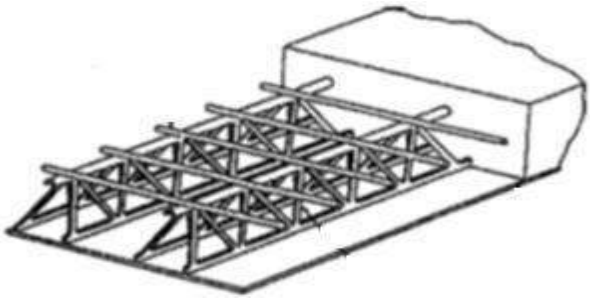

Fig 7.Pyramidal shear connector

vi. RECTANGULAR-SHAPED COLLAR CONNECTORS

This connection device consists of a collar composed of two or more parts, astride the timber beam, bolted together at adjacent wings (Figure 8). At the collar-beam interface, a rubber layer is interposed. The superior wings of the collar or a steel stud, purposely welded to the collar in the upper part, which are immersed in the concrete cast, guarantees the slipping action transmission. The force-slip relationship is used to discuss the connection behavior (Faggiano et al., 2009).

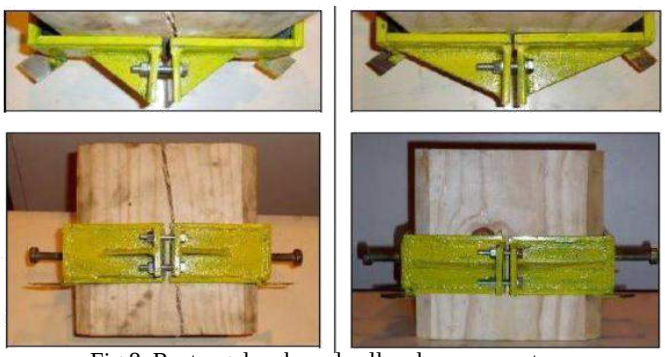

Fig 8. Rectangular shaped collar shear connector

III. ADVANTAGES OF SHEAR CONNECTOR

1. Cost-effective production in large quantities and many different sizes
2. Reliable and safe under static and dynamic stress
3. Mechanical interlock of steel and concrete - lifting-off of concrete slabs is prevented
4. High ductility, considerable increase in bearing capacity through plastic design
5. Anchorage of steel parts in concrete for various load directions, no cracking forces because of mechanical interlock
6. Individual design of reinforcement by pre-planned positioning of steel members in concrete
7. Firm welds produced by stud welding
8. Application by trained operators - no highly qualified welders required
9. Electronically controlled and monitored stud welding equipment ensures repeatability and verifiable quality of the weld.

IV. USES OF SHEAR CONNECTORS

1) Shear connection and transverse reinforcement shall be provided to transmit the longitudinal shear force between the concrete and the structural steel element, ignoring the effect of natural bond between two.
2) Shear connectors may be either of mild steel or high tensile steel irrespective of the grade of steel used in the parent girder. Flexible shear connectors are preferred because of their better performance. Channel, angle, and tee shear connectors may be of mild steel, whereas, the shear studs may be made of high tensile steel.
3) Shear connector shall be generally designed for full transfer of longitudinal shear. Shear connector strength and spacing are to be checked separately for all the limit states using appropriate factored load combinations and factored strength. All shear connectors should be capable of resisting uplift of slab from steel section. channel and stud shear connectors provide adequately safety against uplift. Headed stud shear connectors may be assumed to be provide sufficient resistance to uplift, unless the shear connection is subjected to direct tension, in which case they should be supplemented by anchoring devices.
4) For the verification for ultimate limit states, the shear connectors provided in terms of size and shape may be kept constant over length where the design longitudinal shear per unit length does not exceed the design shear resistance by more than 10% over such length, the total design longitudinal shear force should not exceed the total design shear resistance.

V. CONCLUSION

An attempt has been made to review various types of shear connector in composite structures. The review concludes with a discussion of recent applications of shear connectors in composite structures. Despite being commonly used to transfer longitudinal shear forces across the steel concrete interface, the headed stud shear connectors have some disadvantages and difficulties to be used in composite beams.

- To combine the large strength of a block type connector with some ductility and uplift resistance arising from the holes at the perfobond connector web, T-perfobond connectors were introduced. For similar longitudinal plate geometries, the resistance and stiffness of this type of connector are generally higher than that of the perfobond connectors.
- The load capacity of oscillating perfobond strip connectors when compared to that of the headed studs and T-shape connectors is generally larger. However, due to the fast drop in the load capacity after the peak, it portrays unsatisfactory performance

when used in the case of ordinary strength and normal weight concrete.

- The behavior of the T-connector is very favorable. The beating stress on the front of the T is very high, as a result of the relatively small area. Local concrete crushing occurs, which results in a quasi-plastic performance.
- Channel connectors might not need inspection procedures, such as bending test of headed studs due to the highly reliable conventional welding system used in the welding of these connectors. The load carrying capacity of a channel shear connector is higher than that of a stud shear connector.
- Pyramidal shear connector which is a welding shear connector may reduce the fatigue strength of the thin bottom plate.

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TECH-CHRONICLE
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Comparative Analysis of Vehicular Underpass For Different IRC Live Load

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Abstract—The Underpass RCC Bridge is very rarely adopted in bridge construction but recently the Underpass RCC Bridge is being used for traffic movement. In this paper, the comparative analysis of the vehicular underpass RCC Bridge is carried out. The analysis of underpass RCC Bridge is done by applying spring constant i.e. modulus of subgrade reaction to the raft, calculated assuming the young's modulus of soil. 2D model is prepared considering unit meter width and comparison is made on the basis of design forces i.e. Bending Moment and Shear Forces. In this study we show a percentage difference in design values for new and old IRC loadings. 2D model can be effectively used for analysis purpose for all the loading condition mentioned in IRC:6-2014, "Standard Specifications and Code of Practice Road Bridges" The Indian Roads Congress.

Keywords-RCC Underpass bridge, Spring Constants.

1.Introduction

The Underpass RCC Bridge is very rarely adopted in bridge construction but recently the Underpass RCC Bridge is being used for traffic movement. Main attribute to the design concept were speedy construction, least disturbance to the traffic during construction, enhanced aesthetics, effective drainage and confortable lighting The vehicular underpass may subjected to road traffic (IRC loading) or train traffic (IRS loading), in this paper underpass is analyzed for IRC loadings (IRC:6-2014).
In this paper 2D analysis of underpass RCC bridge is carried out considering different loading conditions and different loading combinations which are considering from IRC:6-2014, "Standard Specifications And Code Of Practice Road Bridges" The Indian Roads Congress. The analysis of underpass RCC Bridge is done by applying spring constant i.e. modulus of subgrade reaction to the raft, calculated assuming the young's modulus of soil as 3000t/m^2.

1.1 Modeling of system

For the study of Underpass RCC bridge, earth pressure acting on side walls of underpass RCC bridge because structure embedded as well as vertical loading due to imposed load and live load on the top of underpass RCC bridge is considered. Also the impact and braking load corresponding to live load is considered as per IRC:6-2014. As there is a top loading, there is reaction at bottom also. Spring constants are applied to the raft calculated from book Bridge Deck Behavior by E.C. Hambly.

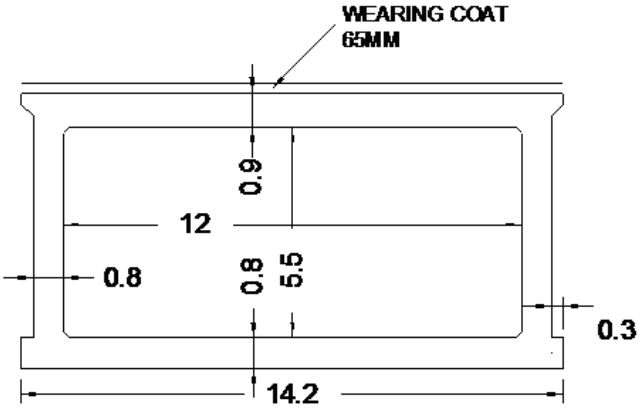

Figure 1: Schematic Diagram of RCC Underpass Bridge

Figure 1 shows the schematic drawing for RCC underpass which is analyzed in STAAD considering different load cases and combinations.

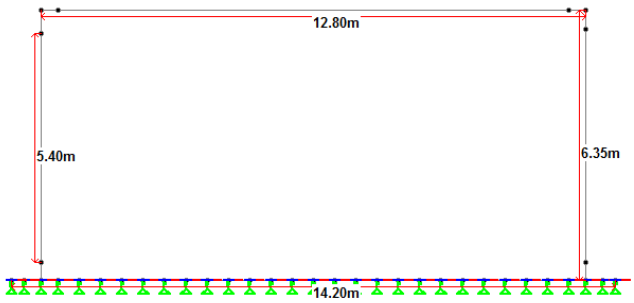

Figure 2: 2D Model of RCC Underpass Bridge
2D underpass RCC bridge model shown in figure 2 is analyzed considering soil structure interaction.

2. Formulation

2.1 Loads on the top of slab

Total load for bending moment and shear force is considered from IRC code rules specifying the loads for designing the superstructure and substructure of bridges and for assessing the strength of existing bridges.
Dead load of box = Area x thickness x density - - 1.1

Total vertical pressure on top slab = Imposed load + Dead load + Live load - - 1.2

2.2 Loads on sidewalls

The coefficient of active earth pressure of the soil is given by the equation

$$Ka = \frac{\cos^2(\emptyset - \alpha)}{\cos^2\alpha \times \cos(\alpha + \delta) \times \left(1 + \sqrt{\frac{\sin(\emptyset + \delta) - \sin(\emptyset - i)}{\cos(\alpha - \delta) - \cos(\alpha - i)}}\right)^2} \quad - - \quad 1.3$$

where,

γ = Density of soil, ϕ = Angle of internal frictional δ= angle of friction between wall and earth fill

Where value of δ is not determined by actual tests, the following values may be assumed.

(i) δ = 1/3 ø for concrete structures.

(ii) δ = 2/3 ø for masonry structures.

i = Angle which the earth surface makes with the horizontal behind the earth retaining structure

(*i* =0 0 for embedded structure).

Since this concrete structure is embedded in soil, the value of δ is considered as 1/3 ø (for concrete structures) considered for calculation of coefficient of active earth pressure of the soil.

2.3 Earth pressure acting on the sidewalls:

2.3. a) Earth pressure due to backfill

Earth pressure center of top slab = *Ka* x γ x *H* --1.4

Earth pressure center of bottom slab = *Ka* x γ x *H* --1.5

2.3. b) Earth pressure due to dead load surcharge

Earth pressure acting on sidewalls:

At Top = Imposed load + Earth pressure on the top of slab + Live load --1.6

AT Bottom = Horizontal effect of surcharge + Earth pressure center of bottom slab --1.7

2.4 Reaction at the bottom of box

Self weight of box = Weight of top slab + Weight of bottom slab + Weight of side walls - -1.8

Total reaction at bottom=Self weight of box +Weight of imposed load +Weight of live load - -1.9

The boundary condition considered is fixed.

3. Analysis of 2D underpass RCC bridge model

A 2D underpass RCC bridge (Figure 2) is modeled considering 1m width for the following details shown below. Box dimensions: 12.8m x 1m x 6.35m (L x W x H) (Center to center). In addition to the dimensions mentioned in Figure 1, following parameters are considered for the 2D analysis. Keeping all the parameters same, the analysis is carried out using STAAD.Pro (V8i) (programming software). The live load position for maximum bending moment at mid-span and at support and shear force at support is worked out by running the live load in STAAD model thought the span. The dispersed load area is calculated as per IRC:112-2011 Annex.B-3. In final model all live load with dispersed load is added with other load in different load combinations as per IRC:6.

Dimension of underpass RCC bridge considered for analysis are as follows:

Side wall thickness,	=	800mm
Clear height of box,	=	5500mm
Clear Span of VUP, ,	=	12000mm
Thickness of deck slab,	=	900mm
Thickness of base slab,	=	800mm
Base slab projection,	=	300mm
Thickness of fill over deck	=	65mm
Idealised span of cell,	=	12800mm
Idealised height of box, H	=	5500 + 900 / 2 + 800 /2
	=	6350mm

Cantilever length of base slab Lc = 300 + 800 /2 = 700mm

Width of super structure b = 8500 mm

(2 lane carriage–way is considered in paper i.e. 7.5m + 0.5m crash barrier on both side)

Thickness of crash barrier = 500mm

The max BM and SF obtained for 2D underpass RCC bridge model considering soil stiffness are shown in Table 1. Shear force and bending moment diagram for dispersed class A load after combining with other load such as DL, earth pressure, Impact, braking is shown in Figure 3 (a) & (b)

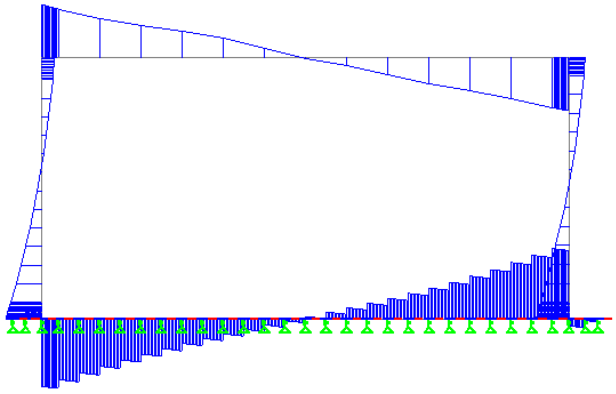

Figure 3: (a) SF diagram for Class A Load

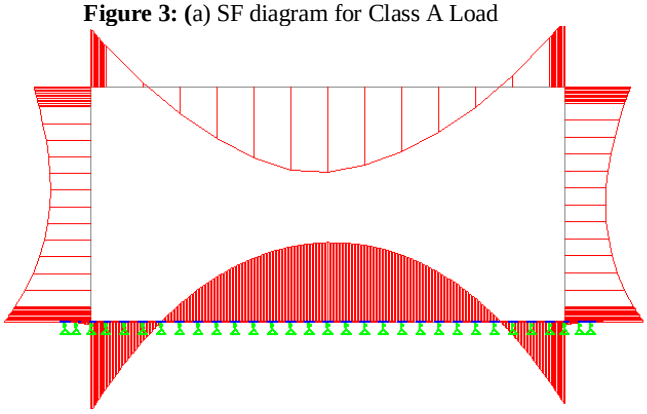

Figure 3: (b) BMD for Class A Load

3.1 Validation of results

The bending moment results obtained by slope deflection method and STAAD program for 2 dimensional model of underpass RCC

bridge are approximately same. The slight variation of results may be due to the variation of moment of inertia values. Based on this validity of results further analysis of same 2D model for various combinations of loading cases was carried out.

4. Comparison of Result of Underpass RCC Bridge Model for Different Live Loads

The comparison of the maximum bending moment and shear force values obtained for different live load cases for 2D underpass RCC bridge models which are considered with soil stiffness are compared. The comparison between newly added Special Vehicle with old vehicles such as class A, 70R trains are made and results are tabulated in Table 1 to 4. The values of bending moment and shear force for 2D model for all loading cases and combinations considered for the analysis purpose from IRC: 6-2014, "Standard Specifications and Code of Practice Road Bridges" The Indian Roads Congress.

Table 1: Comparison of Max BM and Max SF (Class A and Special Vehicle)

Comparison of Max BM & SF for 2 Trains of Class A Vehicle and Special Vehicle Loading				
Members	Design Values	Class A	Special Values	Percentage Difference
Top Slab	BM at mid	45.039	49.430	8.883
	BM at Support	35.337	36.776	3.913
	SF at Support	22.138	28.488	22.291
Bottom Slab	BM at mid	41.400	48.800	15.164
	BM at Support	47.100	52.300	9.943
	SF at Support	29.100	34.000	14.412
Side Wall	BM at mid	21.800	27.000	19.259
	BM at Support	45.400	50.300	9.742
	SF at Support	15.200	14.800	-2.703

Table 2: Comparison of Max BM and Max SF (70R Tracked Vehicle and Special Vehicle)

Comparison of Max BM & SF for 70R Tracked Vehicle and Special Vehicle Loading				
Members	Design Values	70R Tracked	Special Values	Percentage Difference
Top Slab	BM at mid	45.533	49.43	7.88
	BM at Support	36.494	36.776	0.77
	SF at Support	24.4691	28.4876	14.11
Bottom Slab	BM at mid	41.000	48.800	15.98
	BM at Support	47.500	52.300	9.18
	SF at Support	30.700	34.000	9.71
Side Wall	BM at mid	21.900	27.000	18.89
	BM at Support	47.400	50.300	5.77
	SF at Support	15.500	14.800	-4.73

Table 3: Comparison of Max BM and Max SF (70R Wheeled Vehicle and Special Vehicle)

Comparison of Max BM & SF for 70R Wheeled Vehicle and Special Vehicle Loading				
Members	Design Values	70R Wheeled	Special Values	Percentage Difference
Top Slab	BM at mid	49.154	49.43	0.56
	BM at Support	40.805	36.776	-10.96
	SF at Support	29.2201	28.4876	-2.57
Bottom Slab	BM at mid	46.100	48.8	5.53
	BM at Support	52.100	52.3	0.38
	SF at Support	34.000	34	0.00
Side Wall	BM at mid	25.900	27	4.07
	BM at Support	50.700	50.3	-0.80
	SF at Support	15.700	14.8	-6.08

Table 4: Comparison of Max BM and Max SF (70R Boggie Load and Special Vehicle)

Comparison of Max BM & SF for 70R Boggie Load and Special Vehicle Loading				
Members	Design Values	70R Boggie	Special Values	Percentage Difference
Top Slab	BM at mid	45.079	49.43	8.80
	BM at Support	34.451	36.776	6.32
	SF at Support	24.4549	28.4876	14.16
Bottom Slab	BM at mid	39.800	48.8	18.44
	BM at Support	44.700	52.3	14.53
	SF at Support	30.400	34	10.59
Side Wall	BM at mid	21.000	27	22.22
	BM at Support	45.700	50.3	9.15
	SF at Support	15.900	14.8	-7.43

5. Conclusions

From the analysis it can be observed that bending moment and shear force obtained for different live load cases are different and when compared with new IRC load i.e. Special vehicle, it is found that design values for special vehicle are more as compared to other live loads such as Class A, 70R tracked vehicle, 70R wheeled vehicle and 70R baggie load. Hence, in analysis and design of underpass special vehicle shall be considered as per IRC:6-2014 amendment 1.

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A Study of Behaviour of Macro and Micro Hybrid Fibre on Performance of Concrete

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***Abstract*— Fibre reinforcement is commonly used to provide toughness and ductility to brittle cementations matrices. Reinforcement of concrete with an individual type of fibre may improve the desired properties to a limited level. A composite is termed as hybrid, if two or more types of fibres are combined to produce a composite that derives benefits from each of the individual fibres and exhibits the properties of all the fibres in concrete.**

The work carried out for study of the, behaviour of Steel fibre with micro synthetic fibre and macro-synthetic fibres with micro-synthetic fibres to improve the tensile behaviour of fibre reinforced concrete, such that this can be used for the maximum tensile and compressive behaviour of Hybrid fibre reinforced concrete.

Keywords: FRC, Hybrid FRC, Polypropylene Fibre, Macro synthetic Fibre, Micro synthetic Fibre

INTRODUCTION

Fiber Reinforced Concrete (FRC) is concrete containing fibrous material which increases its structural integrity. It contains short discrete fibers that are uniformly distributed and randomly oriented. Fibers include steel fibers, glass fibers, synthetic fibers and natural fibers. Each one of which is having varying properties to the concrete.

The concept of using fibers as reinforcement is not new. Fibers have been used as reinforcement since ancient times. Historically, horsehair was used in mortar and straw in mud bricks. In the 1900s, asbestos fibers were used in concrete. In the 1950s, the concept of composite materials came into being and fiber-reinforced concrete was one of the topics of interest. Once the health risks associated with asbestos were discovered, there was a need to find a replacement for the substance in concrete and other building materials. By the 1960s, steel, glass, and synthetic fibers such as polypropylene fibers were used in concrete. Research into new fiber-reinforced concretes continues today.

Fibers are usually used in concrete to control cracking due to plastic shrinkage and to drying shrinkage. They also reduce the permeability of concrete and thus reduce bleeding of water. Some types of fibers produce greater impact, abrasion, and shatter resistance in concrete. Generally fibers do not increase the flexural strength of concrete, and so cannot replace moment resisting or structural steel reinforcement. Indeed, some fibers actually reduce the strength of concrete.

Luca [1] studied two different types of hybrid composites with steel and polypropylene Macro-Fibres as well as polypropylene and carbon Micro-Fibres. Study indicates that HYFRC can truly be engineered with suitable mix designs for various applications. Zongcai and Jianhui [2] investigated on hybrid fibres including high elastic modulus steel fibre and low elastic modulus synthetic macro-fibre (HPP) as two elements were used as reinforcement materials in concrete. The investigation of Zongcai and Jianhui [2] using high elatic modulus steel fibres and low elastic modulus synthetic macro fiber showed that when the total fibre volume fractions (V_f = *0.5)* were kept as a constant (V_f = 1.5), compared with single type of steel or HPP fibres, hybrid fibres can significantly improve the toughness, flexural impact life and fracture properties of concrete. Milad [3] presents the influence of different fibre in high-performance lightweight concrete and the ductility capacity of reinforced lightweight concrete beam. Tamil and Thandavamoorthy [4] studied the strength of concrete cubes, cylinders and prisms cast using M30 grade concrete and reinforced with steel and polypropylene fibres. Also, hybrid fibres with crimped steel and polypropylene were used in concrete matrix to study its improvements in strength and durability properties.

Machine H., et.al [5] investigates the mechanical properties of Polypropylene hybrid Fibre-Reinforced Concrete. The results showed that the compressive strength, splitting tensile strength, and flexural properties of the polypropylene hybrid fibre-reinforced concrete are better than the properties of single fibre-reinforced concrete. Rashid H., et.al [6] gives contribution towards the flexural properties of metallic-hybrid-fibre reinforced concrete. The test results on hybrid-fibre-reinforced concrete showed that the two metallic fibres when used in hybrid form result in superior performance compared to their single-fibre reinforced counterparts. Harsh [7] studied SFRC which has the ability of excellent tensile strength, flexural strength, shock resistance, fatigue resistance, ductility and crack arrest. Darole J. S., et.al [8] studied Hybrid fibre can provide reinforcement at all the range of strains. Combination of low and high modulus fibres can arrest cracks at micro level as well as macro level. David J. C. (2013) studied the benefits of fibre reinforced concrete (FRC). David focussed on steel FRC subjected to monotonic loads.

In this work investigated the compressive behaviour of macro synthetic structural fibres as reinforcement for concrete elements, in comparison to the behaviour exhibited by steel fibres.

Experimental Work

Casting and testing of concrete cubes, cylinders, beams were done as per IS code recommendations (IS Code 10262: 2009, Concrete Mix Design). The proportioning of concrete mixes consists of determination of the quantities of respective ingredients necessary to produce concrete having adequate, but not excessive, workability and strength for the particular loading and durability for the exposure to which it will be subjected. Emphasis is laid on making the most economical use of available materials so as to produce concrete of the required attributes at the minimum cost. The basic assumption made in mix design is that the compressive strength of workable concrete is governed by the water cement ratio. The concrete mix adopted was M30 concrete with varying percentage of fibres ranging from 0, 0.25, 0.5, 0.75 & 1%. Even though the mix design need not be done for the basic mixes of M20 it was verified by designing it.

The ingredients of the concrete used include: tap water, Cement (Portland Pozzolana Cement), coarse aggregates with diameters in the range of 10-20mm, medium graded with aggregate size within 4-10mm, fine sand and polypropylene micro, Polypropylene macro fibers and steel fibers.

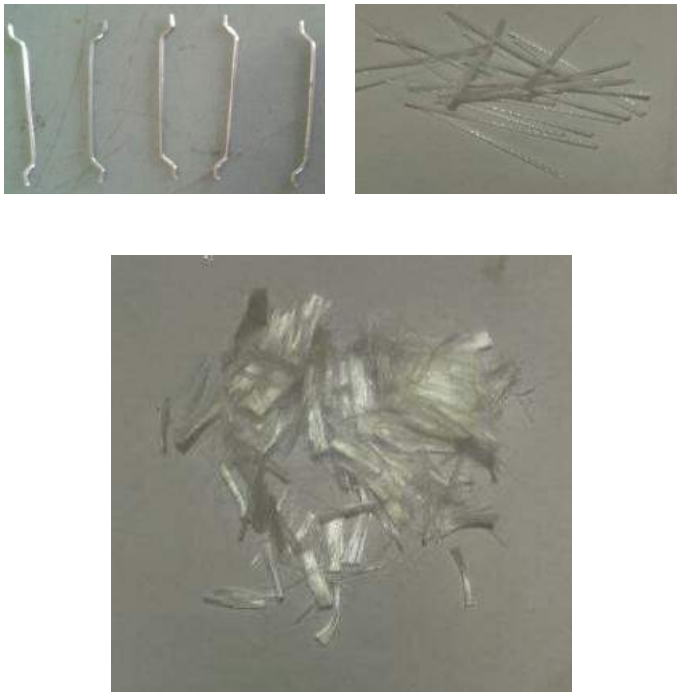

Fig 1: Steel fibers, Macro Polypropylene fibers and Micro Polypropylene fibers

Properties of Fibres

a) Steel Fibre

Type of Fibre:

- Shaktiman MSH 10050 Hook End Steel Fibre

Compliance:

- Conform to ASTM A820, Type I cold drawn wire
- Testing conform with ASTM A820

Nominal dimension:

- Diameter D = 0.7mm
- Length L = 35
- Aspect Ratio L/D = 50
- Hook depth h & h' =1.80mm, +1/-0mm
- Hook length l & l' = 1-4 mm
- Bending angle α & α' = 45^0
- Torsion angle = <30^0
- Tensile strength of wire 1100 MPa
- Strain at failure < 4%

b) Polypropylene Macro Fibre:

Type of fibre

- Embossing fibre

Nominal dimension:

- Diameter D = 0.5mm
- Length L = 35 mm
- Aspect Ratio L/D = 70
- Tensile strength = 30.20 kgf

c) Polypropylene Micro Fibre:

Type of fibre

- monofilament fibre

Nominal dimension:

- Length L = 12 mm
- Diameter D = 30 micron

Tensile strength = 4.5 kgf

Mix Proportions

Six types of concrete cubes were prepared according to mix proportion as Cement : Fine Aggregate : Coarse Aggregate : Water (1 : 1.71 : 2.36 : 0.45). The mix proportion classified as shown in Table 1 according to the use of fiber. The concrete coded as M0 to M5. Concrete M0 stands for the conventional Concrete without fiber and other concrete (i.e. M1, M2 M3, M4, M5) have the variation of the steel fiber, Polypropylene Macro fiber and Polypropylene Micro fibers

Table 1: - Mix Proportion of Concrete Cube

Type of Concrete	% of Steel Fiber	% of Polypropylene Macro Fibre	% Polypropylene Micro Fibre
M0	0	0	0
M1	1	0	0
M2	0	1	0
M3	0.5	0.5	0
M4	0.7	0	0.3
M5	0	0.7	0.3

Scope of Present Investigation

The purpose of this study was to compare the macro synthetic structural fibres as reinforcement for concrete elements, in comparison to the behaviour exhibited by steel fibres.

Test Method

The slump test for all types of concrete mixes was performed with a targeted slump flow of 100 mm±10mm. Table 2 shows the slump of mix. Compressive strength test by casting 150x150x150mm cubes,

Table 2:- Slump of Mix

Type of Concrete	Slump (mm)
M0	100
M1	95
M2	98
M3	96
M4	95
M5	98

Result and Discussion

The test results of Steel fibers, Macro Polypropylene fibers and Micro Polypropylene fibers are compared with conventional Concrete (M0) at 07 and 28 days of curing. The comparison of compressive strength of concrete specimens is given in Table 3.

Table 3 Compressive strength at 7 and 28 days Curing

Sr. No	Mix	Average Strength (7 Days)		Average Strength (28 Days)	
		Obtained N/mm ^{2}	Increased (in %)	Obtained N/mm ^{2}	Increased (in %)
1	M0	23.24	**0.000**	36.74	**0.000**
2	M1	25.25	**8.649**	41.62	**13.283**
3	M2	27.11	**16.652**	37.77	**2.803**
4	M3	26.59	**14.415**	45.18	**22.972**
5	M4	28.15	**21.127**	35.7	**-2.831**
6	M5	28.58	**22.978**	35.85	**-2.422**

From Table 3, the results shows that the % increase in the compressive strength higher with the addition of 0.5 % steel fiber and 0.5% Polypropylene Macro Fibre at 28 days but at 7 days the compressive strength is low as compared to the 0.7% of Polypropylene Macro Fibre and 0.3% Polypropylene Micro Fibre.

Conclusion

Following conclusions are drawn from the present investigation:

- It is evident from the present investigation that the use of various type of fibres at one mix proves to be better as compared to use of unique fibers.
- There was 22% increase in the compressive strength as a result of use of multiple fibers.
- The improved mechanical properties of steel fibers and macro Polypropylene fiber would result in reduction of warping stresses, short and long term cracking and reduction of slab thickness.

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Utilization of plastic waste in Geopolymer concrete

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Abstract: A considerable growth in the production of plastic leading to generation of huge quantity of plastic waste. Reusing this as a sustainable construction material may be an ideal solution. This paper represents some of the recent developments in the field of utilization of waste material for green and sustainable construction material called Geopolymer concrete. It also reveals possibilities of utilization of plastic waste as a constituent, from the work done so far. From the review of past research works it can be concluded that utilization of plastic waste materials shows scope towards development of green concrete which resembles the cement concrete.

1. INTRODUCTION

Since the dawn of human civilization, we are utilizing natural resources for our comfort, betterment and especially to get evolve. In early stages we were utilizing natural resources in its natural form. Due to advancement in science, we forget natural resources as per our needs. When these resources reach to end of their life, we consider them as waste materials.

Today tons of waste is floating all over the world, which leading to waste disposal muddle. Manufacturing industries are growing haphazardly which results in immense amount of organic and inorganic waste. Disposal of these waste is a major problem that we are facing right now. Plastic is one of the most consumptive material all over the world. Plastic was invented in late 20 century. What makes plastic so special is its strength, durability, low cost, ease of manufacture, versatility that also makes it problematic when it comes to its end of life. Plastic waste might be used as a partial or total replacement of fine aggregates as well as course aggregates to obtain desired properties of Geo-polymer concrete.

Utilization or reusing/recycling of these waste as a construction material is a solution to environmental problem. Use of these materials not only helps in getting them utilized in cement, concrete and other construction materials, but also helps in reducing the cost of cement and concrete manufacturing, and also has numerous indirect benefits such as reduction in landfill cost, saving in energy, and protecting the environment from possible pollution effects.

Concrete is being widely used as a construction material all over the world. Generally concrete ingredients are cement, sand and aggregates which are used globally for making concrete. Ordinary Portland cement production is the second major source of generation of carbon dioxide after automobile. Hence, it is necessary to look for an alternative material. Geo-polymer is an innovative, green and sustainable construction material which is produced by the chemical action of inorganic material. Fly ash which is rich in silica and alumina reacted with alkaline solution producing alumina silicate gel that acts as a binding material for concrete. Geo-polymer can be casted without using ordinary Portland cement. Geo-polymer concrete which already is a green and sustainable construction material by blending it with waste material we can achieve an ultimate green concrete. This paper emphasize the constituents of Geo-polymer concrete and its potential application.

1.1 BRIEF OF GEOPOLYMER CONCRETE

Davidovits in 1988 proposed that an alkaline liquid could be used to react with the silicon-(Si) and the aluminium-(Al) in a source material of geological origin or in by-product materials such as fly ash and rice husk ash to produce binders. Due to the chemical action that takes place in this case is a polymerization process, he named the term "Geopolymer" to represent these binders. Geopolymer concrete is concrete which does not utilize any Portland cement in its production. Geopolymer concrete is being widely studied extensively and shows promise as a substitute to Portland cement concrete. Research shifted from the chemistry domain to engineering applications and commercial production of Geopolymer concrete. There are two main constituents of Geopolymer, namely the source materials and the alkaline liquids. The source materials for Geopolymer based on alumina-silicate must be rich in silicon-(Si) and aluminium-(Al). These could be natural minerals such as kaolinite, clays, etc. Alternatively, waste materials such as silica fume, fly ash, rice-husk ash slag, red mud, could be used as base materials. The alkaline liquids are from soluble alkali metals that are mostly sodium or potassium based. The most common alkaline liquid used in geopolymerisation is a combination of sodium hydroxide (NaOH) or potassium hydroxide (KOH) and sodium silicate (Na2SiO) or potassium silicate.

1.2ADVANTAGES OF GPC OVER PCC

Geopolymer concrete is considered to be an innovative material that is a viable alternative to traditional Portland cement concrete. There are numerous advantages of Geopolymer over Plain cement concrete.

1.2.1 High compressive strength:

Geopolymer concrete has higher compressive strength than ordinary Portland cement. It also has a properties of rapid

strength gain and it cures very quickly which makes Geopolymer concrete an excellent option for quick builds. Geopolymer concrete has upper hand over ordinary Portland cement concrete when it comes to resist tensile forces. In other words Geopolymer concrete has high tensile strength and is less brittle than ordinary Portland cement. It also consist higher moment than OPC which makes it suitable for earthquake resistance building.

1.2.2 Very low creep and shrinkage:

Shrinkage may be define as the contracting or shrinking of concrete in its hard form because of loss in capillary water. This shrinkage is a responsible factor for an introduction of tensile stresses in concrete which may cause cracking, deformation of concrete even before the concrete is subjected to any amount of external force. Geopolymer has very less amount of water and have very less pores than OPC which prevents losses of capillary water so it will not experience significant shrinkage Creep is defined as deformation or change in shape of structure subjected to sustained load. Basically, long term pressure or stress on concrete can make it change shape. The creep of Geopolymer concrete is very low.

1.2.3 Resistant to heat and cold:

Generally normal concrete face layer crack and spelling when it comes under excessive temperature. But Geopolymer can withstand at higher temperature without any damage. Geopolymer can withstand up to 1200° Celsius without any sign of damage. As for cold temperatures, it is resistant to freezing. The pores are very small but water can still enter cured concrete. When temperatures dip to below freezing that water freezes and then expands this will cause cracks to form. Geopolymer concrete will not freeze.

1.2.4 Chemical Resistance:

Concrete pore structure and its permeability is a first line of defence against aggressive chemicals. Geopolymer have very little numbers of pores compare to OPC. So it has a very strong chemical resistance. Acids, toxic waste and salt water will have a less effect on Geopolymer concrete. Corrosion is not likely to occur with this concrete as it is with traditional Portland concrete.

1.2.5 Environment friendly:

Geopolymer cement uses very low energy materials, like fly ashes, slags and other industrial wastes and a small amountof high chemical energy materials (alkali hydroxides) to bring about reaction only at the surfaces of particles to act as aglue.

2. LITERATURE REVIEW OF RECENT DEVELOPMENT IN GPC

Several authors have reported their work on Geopolymer concrete.

Muhammad FadhilNuruddin, Sobia Anwar Qazi and Nasir Shafiqinvestigated on Utilisation of waste material in geopolymeric concrete, they proposed that the flexural strength of geopolymeric concrete was comparable to OPC concrete. External exposure curing conditions were found to be the best curing regime for geopolymeric concrete given that it showed the highest value of compressive strength in comparison with similar concretes cured under ambient curing in their study. The compressive strength of the geopolymeric concrete under external exposure conditions developed up to 28 days however; there was no significant increase in strength beyond this time. The best performing geopolymeric concrete mix design, based on the results of compressive strength tests and a microstructure study, was the concrete mix cured under external exposure conditions. This mix comprised

Sr. No.	Materials	Mass	Unit
1	Fly Ash	350	kg/m3
2	Sodium Hydroxide (NaOH)	41	kg/m3
3	Sodium silicate (Na2SiO3)	103	kg/m3
4	Table sugar	10.5	kg/m3
5	Extra water	35	kg/m3
6	Fine aggregate	645	kg/m3
7	Coarse aggregate	1200	kg/m3

H.R. Prajapati and their fellows worked on Aspects of durability of Geopolymer concrete containing metallized plastic waste their study include The performance of Geopolymer concrete was studied, using test of oxygen permeability and water sorptivity. In oxygen permeability test ingress of gases and water vapour occurs while in water sorptivity test ingress of water and hazardous liquid occurs. Experimental work was done on fly ash and waste metalized polymer plastic based Geopolymer concrete by different sodium silicate to sodium hydroxide ratio 1, 2 and 3, different molar content of sodium hydroxide 8molar, 10molar and 16molar , percentage of metalized polymer plastic waste 0%, 0.5%, 1.0% and 1.5% and cured with hot air at 100°C for 24hr. they concluded that Molar content of Sodium hydroxide (NaOH) when increased from 8M to 16M then Oxygen permeability and Water sorptivity of Geopolymer concrete decreases. Sodium silicate (Na2SiO3) to Sodium hydroxide (NaOH) ratio when increased from 1 to 3 then Oxygen. permeability and Water sorptivity of Geopolymer concrete decreases. Metalized polymer plastic waste increased from 0% to 1.5% and then Oxygen permeability and Water sorptivity of Geopolymer concrete decreases.

M. I. Abdul Aleem and P. D. Arumairaj studied on Geopolymer concrete in their paper they concluded that User-friendly Geopolymer concrete can be used under conditions similar to those suitable for ordinary Portland cement concrete. These constituents of Geopolymer Concrete shall be capable of being mixed with a relatively low-alkali activating solution and must be curable in a reasonable time under ambient conditions. The production of versatile, cost -effective Geopolymer concrete can be mixed and hardened essentially like Portland cement. Geopolymer Concrete shall be used in repairs and rehabilitation works. Due to the high early strength

Geopolymer Concrete shall be effectively used in the precast industries, so that huge production is possible in short duration and the breakage during transportation shall also be minimized. The Geopolymer Concrete shall be effectively used for the beam column junction of a reinforced concrete structure. Geopolymer Concrete shall also be used in the Infrastructure works. In addition to that the Fly ash shall be effectively used and hence no landfills are required to dump the flash. The government can make necessary steps to extract sodium hydroxide and sodium silicate solution from the waste materials of chemical industries, so that the cost of alkaline solutions required for the Geopolymer concrete shall be reduced.

D. Hardjito and their fellows published an invited paper, Brief review of development of Geopolymer concrete in American Concrete Institute, Los Vegas. The authors' experimental results show that the H_2O-to-NaO molar ratio in the mixture composition is a significant parameter affecting the compressive strength of fly ash-based Geopolymer concrete,whereas the influence of the Na_2O-to-SiO_2 molar ratio is insignificant. As the H_2O-to-NaO molar ratio increased the compressive strength of Geopolymer concrete decreased. Also, the compressive strength decreased when the water-to Geopolymer solids ratio by mass increased. Note that the total mass of water in the mixture is the sum of water contained in the sodium silicate solution, the mass of water in the sodium hydroxide solution, and the mass of extra water, if any, added to the mixture. The mass of Geopolymer solids in the mixture is the sum of the mass of fly ash, the mass of sodium hydroxide flakes, and the mass of solids in sodium silicate solution. Geopolymer concrete shows significant potential to be a material for the future, because it is not only environmentally friendly but also possesses excellent mechanical properties, both in short term and long term, and durability.

DjwantoroHardjito and their research teams studied on Geopolymer concrete: Turn waste into environment ally friendly concrete they represented experimental data to show the influence of various salient parameters on the compressive strength of Geopolymer concrete. Limited test results also reveal that Geopolymer concrete undergoes very little drying shrinkage an d moderately low creep, and possesses excellent resistance to sulphate attack. They also pinpointed that Geopolymer concrete is an environmentally friendly and energy-efficient construction material with an enormous potential in many infrastructure applications. The binder in this concrete is produced by synthesizing waste materials that are rich in silicon and aluminium such as low calcium (class F) fly ash.

Er Bharat Bhushan Jindal briefly studied Geopolymer concrete in their paper they mentioned that Fly ash-based Geopolymer is better than normal concrete in many aspects such as compressive strength, exposure to aggressive environment, workability and exposure to high temperature. Study shows that Geopolymer concrete is more resistant to corrosion and fire, has high compressive and tensile strengths, and it gains its full strength quickly (cures fully faster). It also shrinks less than standard concrete. Thus, owing to these structural advantages it may be concluded that in near future Geopolymer concrete may find an effective alternate to standard cement concrete.

N A Lloyd and B V Rangan published a paper on Geopolymer concrete: A review of development and opportunities, This paper presents the results from studies on mix design development to enhance workability and strength of Geopolymer concrete. The influence of factors such as, curing temperature and regime, aggregate shape, strengths, moisture content, preparation and grading, on workability and strength are presented. The paper also includes brief details of some recent applications of Geopolymer concrete. They pin pointed that Basic mixture proportions characterized by 75% aggregate to total mass, alkaline liquid to fly ash of 0.35 (analogous to water to cement ratio) and elevated temperature curing results in a high strength Geopolymer concrete. Ambient curing of Geopolymer has been trial edand further mixture trials with ambient curing are presently being researched. Temperature specification for curing should be correlated to actual specimen temperature for high and very high strength Geopolymer concretes, monitoring temperature may be warranted
if strength is critical and when steam curing, placement of the steam vents or hoses and control thermocouples as well as specimens is important. The introduction of a rest day, that is ambient curing for 24 hours prior to steam curing, resulted in elevated compressive strengths of the order of 20%. As with Portland cement concrete, strength was increased and workability and ease of compaction decreased with a reduction in added water. Strength gain at one day is around 80% of the 28 day strength when cured for 24 hours. As with Portland cement concrete, the aggregate moisture content can be accommodated by adjusting the total water added to a Geopolymer concrete mixture without sacrificing strength or workability. Additionally, the effect of aggregate particle shape and grading on the properties of Geopolymer concrete is similar to that of Portland cement concrete. The paper presented brief details of Geopolymer precast concrete products. The economic benefits and contributions of Geopolymer concrete to sustainable development are also outlined. Mixdesign for same shown in following table.

Table: 2 Mix design for Geopolymer concrete

Material	Nominal 40MPa mixture(kg /m3)	Nominal 60MPa mixture(kg/ m3)	Nominal 75MPa mixture(k g/m3)
20 mm aggregate	641	641	641
7 mm aggregate	641	641	641
Sand	549	549	549
Flyash	404	404	404
NaOH solution	41	41	41
$Na2SiO_3$solu	102	102	102

tion			
Super Plasticizer	6	6	6
Added water	25.5	17	13.5

Zainab Z. Ismail and Enas A. AL-Hashmi worked on Use of waste plastic in concrete mixture as aggregate replacement, they have mentioned that, the compressive strength values of all waste plastic concrete mixtures tend to decrease below the values for the reference concrete mixtures with increasing the waste plastic ratio at all curing ages. This may be attributed to the decrease in the adhesive strength between the surface of the waste plastic and cement paste. In addition waste plastic is hydrophobic material which may restrict the hydration of cement. The dry density values of waste plastic concrete mixtures at each curing age tend to decrease below values for the reference concrete mixture, but they remain averaged to that of the reference concrete mixtures. At 28 days curing age, the lowest dry density (2223.7 kg/m3) exceeds the range of the dry density of structural lightweight concrete. The slump values of waste plastic concrete mixtures showed a tendency to decrease below the slump of the reference concrete mixture. In spite of this decline in the slump of those mixtures, those mixtures are easyto work based on the consideration that workability has a broad range from very low to high workability for different applications, The flexural strength values of waste plastic concrete mixtures tend to decrease below the values for the reference concrete mixtures with increasing the waste plastic ratio. A concrete mixture made of 20% waste plastic has the lowest flexural strength at 28 days curing age, viz. 30.5% below the
value of the reference concrete mixture.

R. A. Patel, AnkurBhogayta and their fellows worked on Flexural response of Geopolymer concrete beam containing metalized plastic waste, the beam was analysed using stress strain behaviour model proposed by Collins et. al. (Structural Design Considerations for High Strength Concrete, ACI concrete international 15(5):27-34) and compared with testresults.They have highlighted that as the analytical model which is applicable to reinforced concrete given by Collins is able to predict ultimate flexural strength of Geopolymer concrete containing metallized plastic waste, indicating its flexural response is similar to reinforced concrete.For balance reinforced beam moment resistance capacity was nearly same in analytical and experimental.For analytical study moment resistance capacity was decreased 12.92%, 4.54% and 3.72% in shear reinforced beam, over reinforced beam and under reinforced beam respectively as compare to experimental.

AnkurBhogayta and their fellows worked on Strength Properties of Concrete Containing Post Consumer Metalized Plastic Wastes in their experimental work, conventional concrete with 0.45 water/cement ratio was prepared and plastic pellets were added by 0.5%, 1% and 1.5% by concrete volume. Compressive strength, split tensile strength and surface tensile test were performed on the samples. The test results reviled the reduction of strength properties up to 60%. It could be observed that the plastic beyond 1.5% proportion could make the concrete not suitable for construction work except where the lean concrete could be used. They have also added that, the most suitable water to cement ratio was found as 0.45. It could be noticed that increase in water to cement ratio reduces the cement content and ultimately more reduction of basic strength properties could be expected.

3. DISCUSSION

Based on various papers, it is observed that there is significant advantage in the usage of fly ash in concrete for the development of environmental friendly and sustainable concrete called Geopolymer concrete, which obtained from partially or fully replacement of Portland cement in concrete, while maintaining excellent mechanical properties with enhanced durability performance. As Geopolymer concrete is a small step towards green concrete, by blending it with plastic waste might be one step forward towards ultimate green concrete which shall prove healthy to environment. Plastic waste might be utilized as a partial or total replacement of fine aggregates, coarse aggregates or could be added as filler materials.

4. CONCLUSION

Based upon above literature review it can be concluded that over the last decades, satisfactory research has been conducted on mechanical, chemical and durability aspects of Geopolymer concrete varying in proportions, curing temperature, curing time and additives. Also very satisfactory work has been conducted on different aspects of cement concrete, blended with plastic waste. However, a need of explicit data is felt from the literature review on utilization of plastic waste in Geopolymer concrete.

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To Study the Optimization of Construction Machinery use for Construction Project

A Short Review

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Abstract— The largest share of investment in all developing countries finds its way into construction. In India construction industry is growing rapidly. So to maintain optimum utilization of equipment is necessary. Construction equipment plays a significant role in the execution of modern high-cost time-bound construction projects. Optimization of equipment contributes to economy, speed and timely completion of a project. Equipment is a capital investment and the contractors must apply time value analytical formula to the process of machine utilization.

The scope of this study is restricted to RMC and Hydraulic Excavator; considering the extensive use of these equipments in modern construction project, wide spread area of factors affecting the hourly output and the time limit for such a study. And different practices of improving the productivity of construction for the Indian scenario. With the help of literature review.

Keywords—Equipment Planning; Factors affecting,

I. INTRODUCTION

The construction industry is best described as an amalgam of wide varieties of economic activities ranging from house building and repair to major engineering works. Drawing the boundaries of construction industry is an arduous task but an operational definition which recognizes the construction industry to include firms and individuals involved in planning, design, the supply of building materials, plant, equipment, transport and other services relating to the procurement of physical infrastructure and services. Thus, the construction industry is a very diversified and dynamic industry. However, this definition reflects the attributes of an effective and functioning construction industry that is well integrated with the wider economy.

II. SCOPE OF STUDY

The assessment of equipment performance needs experience. The equipment capability to perform an assigned task under a given situation can best be determined from the on-site actual trials or it can be accessed from its past performance records of operation under similar site conditions. The main factors which affect the performance of equipment / plant and for which allowance must be made when estimating output are of two types,

1. Factors affecting hourly output
2. Factors affecting overall output

The scope of this paper is to evolve output with regard to factors affecting hourly output. Output can be defined as the average output given by particular equipment where as the output norm is the maximum output the equipment can achieve with the same job conditions. For planning purpose we will be depending on the output of equipment rather than output norms. The norms can be used for increasing the production or for introducing incentive scheme for workers.
Earth moving work being an important activity for any kind of civil projects, the output study of Hydraulic Excavator becomes a role model for other earth moving machinery. The soil condition is the most predominant factor for production in case of excavator and same is the case with dozer, scraper, clamshell, dragline etc. Now a day's hydraulic backhoe excavators are extensively used for earthwork excavation and pipe laying works. More importantly the earthwork is the first activity for any project and other activities are dependent on this. Here the planning managers will have to allocate the resources to complete the particular sector of earthwork in optimum time Again these excavators are used at mining and quarrying works. The factors affecting the output of excavators are numerous and such a study can help in identifying the factors and for assuming realistic production figures.

Concrete Batching plants are a necessary for large construction projects involving huge concreting volume. The ready mix plants are acquiring more popularity now a days, meeting the concrete requirements for commercial and residential projects coming up in town areas. These plants can give well mixed, consistent quality of concrete in less time compared with conventional drum mixers. The study of actual output given by these plants can give a realistic picture

compared to what manufactures has specified as the output. Regarding batching plants, the factors affecting the output are rather manageable as requirement of concrete at site, arrangement for transporting the concrete to placing point etc. Here also the factors affecting the output can be compared with other plants used for general construction projects.

III. OBJECTIVE OF STUDY

The construction industry is basically depending on the labors output. But now a day as the construction industry is growing at a very fast rate and plays a vital role in the development of the nation. This requires the optimum use of the construction equipment's in the execution.

IV.LITERATURE REVIEW

A. HYDRAULIC EXCAVATORS

This is a basic earthmoving machine which can be used for different works with different types of front & back attachments. A basic shovel has the means of propulsion of the machine, of revolving the superstructure around and of operating the head attached to it. Hydraulic excavators are classified by digging motion of the boom and stick to which the bucket is attached. It consists of the following,

1. Basic component providing locomotion and power.
2. Revolving superstructure and mounting. Mounting are of three types,
   i) Crawler mounted
   ii) Wheel mounted
   iii) Truck mounted
3. Attachments to suit the work in hand.

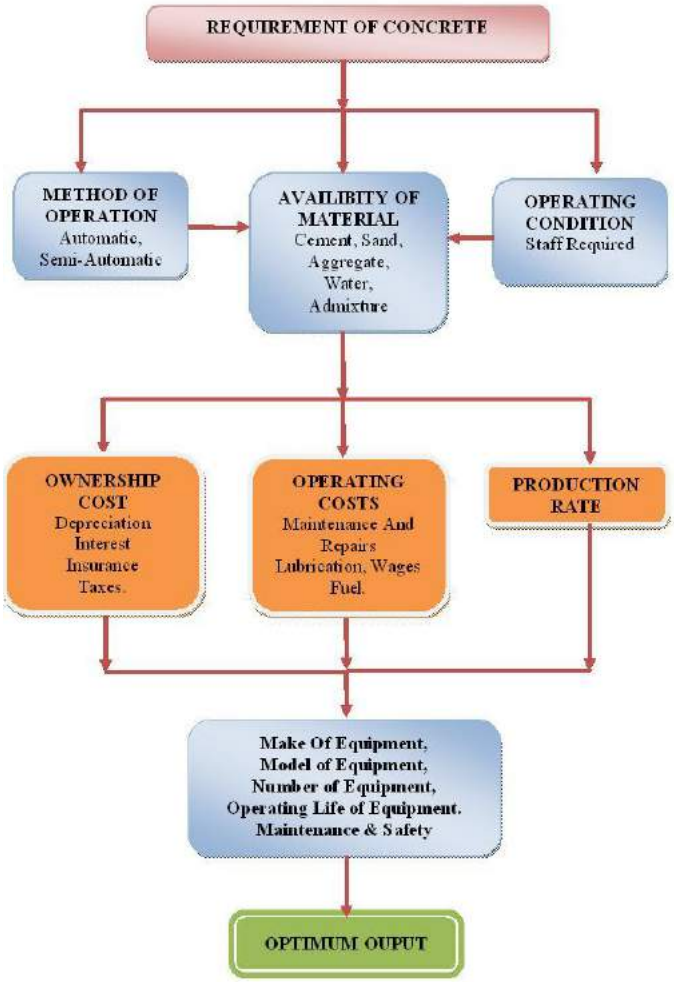
Hydraulics is heart of excavator. Available hydraulic power is important, rather than engine power. Higher engine power only makes it possible to maintain the available hydraulic power at high altitudes. Hydraulic power is a function of pressure and flow rate. Hydraulic power is used to operate digging attachment-boom, stick and bucket-swing gear and track drive. This is achieved either by independent power supply, parallel type power supply or series type power supply.

B. READY MIX CONCRETE PLANT

Batching Plants may be static as well as mobile. Permanent or static plants take a considerable amount of time for erection. Portable plants can be easily transported or hauled, but time may still be required to prepare for the move. Mobile plants are furnished with their own wheels, so that tractor may be connected to the plant to tow it along the road from one site to another. Concrete batching plants, used for manufacturing controlled concrete, may be divided into types based on several criteria. They may be classified as central mix, transit mix or dry / wet batch plants depending upon the application. They may be mass concrete, paving, ready mix or concrete

product plant depending on ultimate use for which the plant is intended. They may be also classified on the basis of their mobility as permanent, portable or mobile.

The aggregate may be put into a stockpile or directly in a storage bin by the use of belt conveyor, bucket elevators, or clamshell buckets. Cement is usually placed into a watertight storage silo or bin by pneumatic pumping or by a screw conveyor and bucket elevator. Water and the liquid admixtures are pumped to a storage vessel. Aggregate bucket elevators may be used to charge the aggregate storage. These are normally used only when space is not available for a belt conveyor. The control provided now days is usually fully automatic.



Showing Conceptual Diagram For R.M.C

Control system vary from the simple hand-lever type to the highly sophisticated electronic controls with which the operator merely selects the mix design, selects the number of cubic meter desired and pushes the start button. This control system then produces the material required for the concrete and records the weight of each material as well as the time, date mix, identification number, truck number and any other information which is required. Automatic controls normally

produce more consistent and more reliable results than manual control

Growth of Construction Equipments by value 1940-2000

.

Table I Showing Growth of Construction Equipments

SR.No	Year	Machines (Nos.)	Value (Crores)
1.	1940	2	0.0074
2.	1945	2	0.0040
3.	1950	95	0:421
4.	1955	238	1.62
5.	1960	934	12.04
6.	1965	2197	38.71
7.	1970	643	21.06
8.	1979	16048	297.54
9.	1984	N.A.	408.44
10.	1998	N.A.	6600.00
11.	1999	N.A.	8000.00
12.	2000	N.A.	9870.00

Government, though belatedly, has assigned the topmost priority for development of infrastructure in the country and the result is the launching of numerous ambitious projects such as Golden Quadrilateral and North – South and East-West Corridor under National Highways Development Project (NHDP), Pradhan Mantri Gram Sadak Yojana (PMGSY) and the River Interlinking to name a few. Besides, Government has taken initiatives for private sector participation, loan assistance from external agencies, like, World Bank, Asian Development Bank, Japan Bank for International Cooperation.

This goal can be achieved only by mechanized construction as only modern high performance construction machinery can ensure process control in real time for achieving end product quality conforming to design specifications. For speedy construction works to complete the project within a fixed time frame, deployment of adequate and appropriate machinery has to be ensured. Moreover, mechanized construction aims at technology up gradation.

According to Seung C. ok & Sunil K.Sinha, Estimating equipment production rates is both an art and a science. An accurate prediction of the productivity of earthmoving equipment is critical for accurate construction planning and project control. Owing to the unique work requirements and changeable environment of each construction project, the influences of job and management factors on operation productivity are often very complex. Hence, construction productivity estimation, even for an operation with well-known equipment and work methods, can be challenging. This study develops and compares two methods for estimating construction productivity of dozer operations (the transformed regression analysis, and a non-linear analysis using neural network model). It is the hypothesis of this study that the proposed neural networks model may improve productivity estimation models because of the neural network's inherent ability to capture non-linearity and the complexity of the changeable environment of each construction project. The comparison of results suggests that the non-linear artificial neural network (ANN) has the potential to improve the equipment productivity estimation model.

According to Russell S. Jeffrey, Recently, constructability has received considerable attention from researchers and practicing engineers. Constructability has been defined as the optimum use of construction knowledge and experience in planning, design, procurement, and field operations to achieve overall project objectives ("Constructability" 1986). This paper discusses the evolution of constructability and how programs have been developed to bring design and construction closer to the level of integration once achieved by the master builder. There is a great deal of discussion among industry professionals as to how constructability is related to total quality management and value engineering. This paper conceptually describes these interrelations. In addition, the paper presents a framework to measure costs and benefits related to constructability. By providing owners with this framework, the parameters will be visible and defined, thus removing skepticism as to the measurement process as well as enabling more consistent and uniform results to be obtained. Additionally, these standardized parameters may facilitate developing a means to measure company and industry performance.

V. CONCLUSION

Construction equipment plays a significant role in the execution of modern high-cost time-bound construction projects. Large scale construction activities like road making, construction of railway routes, canals, irrigation works, power generation dams, development of building sites, airports & harbors are clear indicators of growth of a developing country, But in our country population is more even though construction industry are suffering from the labour problems.

Equipment is an economic investment and the contractors must apply time value analytical formula to the process of machine utilization. The proof of how well the planners understands the work and co-ordinate the use of companies equipment is in the bottom line when the contract is completed – at a profit or loss. The most important aspect of estimating and building a construction project is determining production and cost and controlling both during the progress of work.

This study emphasizes the vital role of construction equipment in construction industries to optimize the profit by reducing the labour hours and also aimed to identifying the best practices of improving the productivity of construction for the Indian scenario. With this aid planning would be more systematic, logical and efficient as resource allocators and managers would be aware of output for particular equipment

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Sesismic Stability Analysis Of Various Geometrical Shaped RC Buildings

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Abstract—Irregularities are not avoidable in construction of buildings; however, the behavior of structures with these irregularities during earthquake needs to be studied. Adequate precautions can be taken. A detailed study of structural behavior of the buildings with irregularities is essential for design and behavior in earthquake. The main objective of this study is to understand different irregularity due to plan irregularity and to analyses H shape, T shape, U shape, L shape and rectangular shaped building while earthquake forces acts. This study was initiated to quantify the effect of different degrees of irregularity on Structures designed for earthquake using simplified analysis. The realistic structure could have been different beam span, different loading on beam & different sizes of column. As in this project it was intended to understand basically the behavior of structure as a whole or in part it was thought of choosing structure with frames 4m x 3.5m. It is proposed to check the impact of plan irregularity to select a suitable structural framing system for the analysis and design of multi-storied buildings. Nodal displacements and drifts are determined by performing the linear static analysis. From the design results amount of reinforcement is determined and compared. Frame wise observations from the analysis and design are observed in detail. Number of ten storied moment resisting frames both regular and irregular in plan are analyzed and designed for the present study.

Keywords—Irregular Buildings, Rectangular Shaped Building, L- Shaped Building, U- Shaped Building, H- Shaped Building, T- shaped Building, Base Shear, Displacment, Story Drift, Area of Steel, Shear Force, Axial Force, Bending Moment, Etc.

CHAPTER 1
INTRODUCTION:

Performance of any building frame is said to be good in an earthquake when a building should possess four main attributes, namely simple and regular configuration, adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation, suffer much less damage than buildings with irregular configurations. A building shall be considered as irregular for the purposes of this study, if at least one of the conditions given in Tables 4 and 5 of IS 1893:2002 is applicable.

Many buildings in the present scenario have irregular configurations both in plan and elevation, which in future may subject to devastating earthquakes. In case, it is necessary to identify the performance of the structures to withstand against disaster primarily due to earthquake. Irregularities are not avoidable in construction of buildings; however, the behavior of structures with these irregularities during earthquake needs to be studied. Adequate precautions can be taken. A detailed study of structural behavior of the buildings with irregularities is essential for design and behavior in earthquake. Several related studies have focused on evaluating the response of "regular" structures. However, there is a lack of understanding of the seismic response of structure with irregularities. Therefore, a comprehensive evaluation of the effect of vertical and horizontal irregularities on the seismic demand of building structures is greatly needed.

CHAPTER 2
METHODOLOGY AND MODELING

2.1 GENERAL

The following basis has been considered for modelling of buildings.

1. The building will be used for residential purpose, as an apartment. So that there are no walls inside the building. External walls 230 mm thick.
2. The main beams rest centrally on columns to avoid local eccentricity.
3. For all structural elements, M25 grade concrete will be used.
4. Sizes of all columns in all floors are kept the same.
5. The floor diaphragms are assumed to be semi rigid to have actual effect of floors slab.
6. Centre-line dimensions are followed for analysis and design.
7. Preliminary sizes of structural components are fixed by experience.
8. For analysis purpose, the beams are assumed to be rectangular so as to distribute
   Slightly larger moment in columns.
9. Seismic loads will be considered acting in the horizontal direction (along either of the two principal directions) and not along the vertical direction, since it is not considered to be significant.
10. All dimensions are in mm, unless specified otherwise.

2.2 CASE CONSIDERATION

Problem for analysis of Rectangle, T-Shaped, U-Shaped, L-Shaped & I-Shaped building is consider on the following basic data.

3.2.1 Modeling and analysis

The design parameters are as follows:

Live load	: 25% of 3.0 KN/m^2 at typical floor &
	: 0.0 KN/m^2 on terrace
Floor finish	: 1.0 KN/m^2
Earthquake load	: As per IS-1893 (Part-I) - 2002
Type of soil	: Type II, Medium as per IS: 1893
Story height	: Typical floor 3.2 m.
Floors	: G.F. + 09 upper floors.
Wall thickness	: 230 mm thick brick masonry wall
Column size	: 400X700 mm
Main Beam size	: 400X700 mm

2.2.2 Material Properties

i) Concrete

All components are modelled with concrete grade M25 unless specified in analysis.

For Grade of concrete M 25

$Ec = 5\,000 \sqrt{fck}$ N/mm^2

$= 5\,000 \sqrt{fck}$ MN/m^2

$= 25000$ N/mm^2 = 25000 MN/m^2.

ii) Steel

HYSD bar of Fe 415 confirming to IS: 1786 is used throughout.

2.2.3. Various geometrical models of cases analyzed

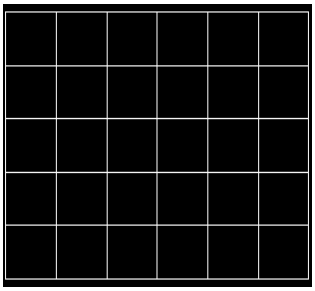


Fig 1.1Plan of Rectangular Shaped Building.

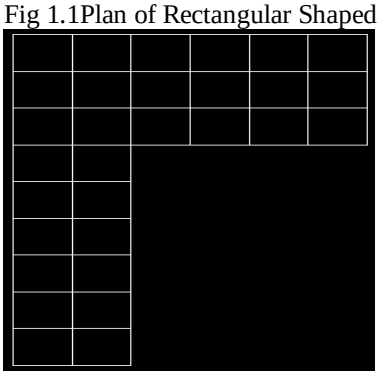


Fig 1.2 Plan of L- Shaped Building.

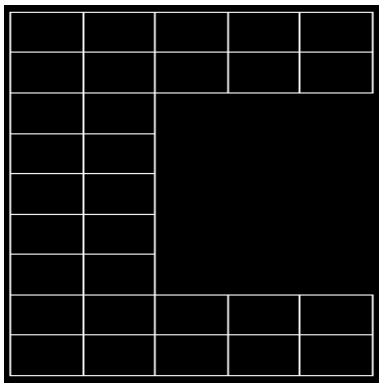


Fig 1.3Plan of U- Shaped Building.

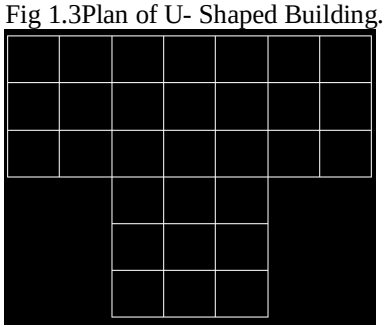


Fig 1.4Plan of T- Shaped Building.

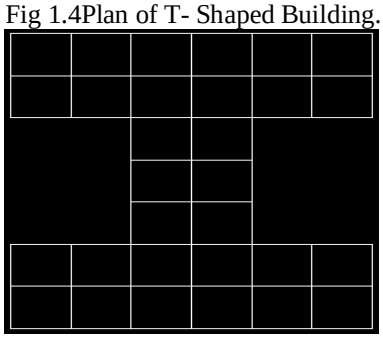


Fig 1.5Plan of H- Shaped Building.

Following are the observations obtained from analysis of Rectangle, T-Shaped, U-Shaped, L-Shaped & I-Shaped building

OBSERVATION TABLE:

Table 1.1 Base Shear summery for all the types of building along X-direction

FLOOR N0.	R-SHAPED BUILDING	T-SHAPED BUILDING	I- SHAPED BUILDING	L-SHAPED BUILDING	U-SHAPED BUILDING
10	634.229	636.405	691.089	669.788	596.057
9	816.188	1042.085	1007.501	1055.05	970.847
8	644.889	823.376	796.05	833.62	767.089
7	493.743	630.397	609.476	638.24	587.302
6	362.75	463.149	447.778	468.911	431.487
5	251.91	321.631	310.957	325.633	299.644
4	161.222	205.844	199.013	208.405	191.772
3	90.688	115.787	111.945	117.228	107.872
2	40.306	51.461	49.753	52.101	47.943
1	54.781	12.865	12.438	13.025	11.986

Table 1.2 Base Shear summery for all the types of building along Z-direction

FLOOR N0.	R-SHAPED BUILDING	T-SHAPED BUILDING	I-SHAPED BUILDING	L-SHAPED BUILDING	U-SHAPED BUILDING
10	526.233	561.421	609.679	767.612	749.083

9	677.208	919.302	888.818	1209.142	1220.092
8	535.078	726.362	702.276	955.371	964.023
7	409.669	556.121	537.68	731.456	738.08
6	300.981	408.578	395.03	537.396	542.263
5	209.015	283.735	274.326	373.192	376.572
4	133.769	181.59	175.569	238.843	241.006
3	75.245	102.145	98.758	134.349	135.566
2	33.442	45.398	43.892	59.711	60.251
1	8.361	11.349	10.973	14.928	15.063

Table 1.2 Base Shear summery for all the types of building along Y- Direction

The variation of base shear in building with no. of story is shown below.

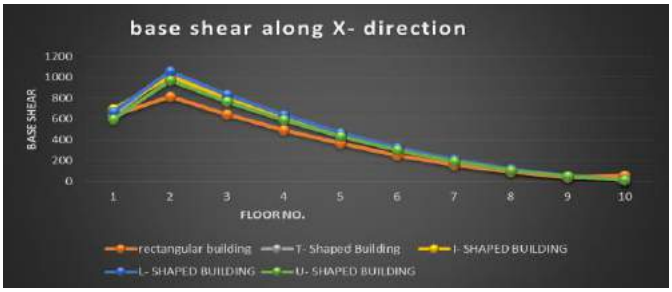


Fig. 2.1 Base shear along X direction

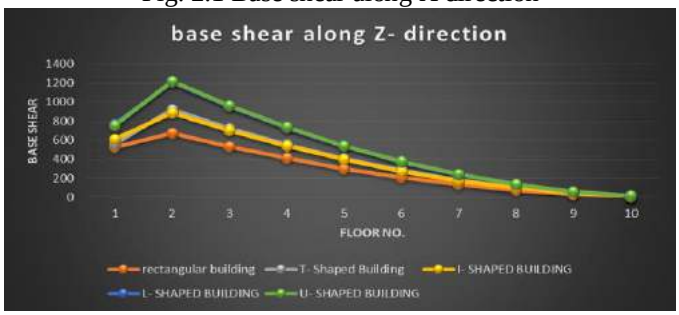


Fig 2.2 Base shear along Z direction

Table 2.1 Lateral Displacement Summery for different load combination along +X Direction for load combination 1.5(DL+(+X))

Lateral Displacement for load combination 1.5(DL+(+X))					
Floor No	R - Shaped	H - Shaped	L- Shaped	T- Shaped	U- Shaped
1	0.097	0.107	0.131	0.129	0.125
2	0.24	0.272	0.323	0.326	0.323
3	0.39	0.446	0.523	0.535	0.539
4	0.538	0.621	0.72	0.746	0.76
5	0.681	0.791	0.911	0.953	0.978
6	0.816	0.951	1.09	1.148	1.188
7	0.936	1.095	1.249	1.325	1.381
8	1.035	1.217	1.381	1.477	1.537
9	1.107	1.311	1.48	1.596	1.688
10	1.154	1.374	1.537	1.681	1.791

Lateral Displacement for load combination 1.2(DL+LL+(+X))					
Floor No	R - Shaped	H – Shaped	L -Shaped	T- Shaped	U - Shaped
1	0.077	0.085	0.105	0.103	0.1
2	0.192	0.217	0.259	0.261	0.259
3	0.312	0.357	0.418	0.428	0.431
4	0.43	0.497	0.576	0.597	0.607
5	0.545	0.633	0.729	0.762	0.782
6	0.653	0.761	0.871	0.918	0.95
7	0.749	0.876	0.999	1.06	1.104
8	0.828	0.974	1.105	1.182	1.24
9	0.885	1.049	1.184	1.277	1.35
10	0.923	1.1	1.229	1.345	1.433

Table 2.2 Lateral Displacement Summery for different load combination along +X Direction for load combination 1.2(DL+LL+(+X))

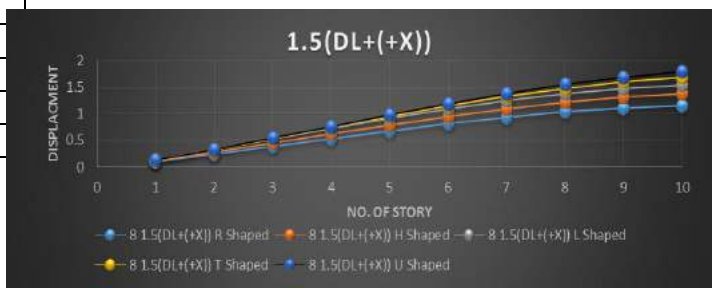


Fig. 3.1 Lateral Displacement for load combination 1.5(DL+(+X))

Fig. 3.2 Lateral Displacement for load combination 1.2(DL+LL+(+X))

Story Drift calculation for load combination 1.5(DL+(+X))					
Floor No.	R - Shaped	H – Shaped	L -Shaped	T- Shaped	U - Shaped
1	0.143	0.165	0.192	0.197	0.198
2	0.143	0.165	0.192	0.197	0.198
3	0.15	0.174	0.2	0.209	0.216
4	0.148	0.175	0.197	0.211	0.221
5	0.143	0.17	0.191	0.207	0.218
6	0.135	0.16	0.179	0.195	0.21
7	0.12	0.144	0.159	0.177	0.193
8	0.099	0.122	0.132	0.152	0.17
9	0.072	0.094	0.099	0.119	0.137
10	0.047	0.063	0.057	0.085	0.103

Table 3.1 story drift Summery for different load combination along +X Direction for 1.5(DL+(+X))

Table 3.2 story drift Summery for different load combination along +X Direction for 1.2(DL+LL+(+X))

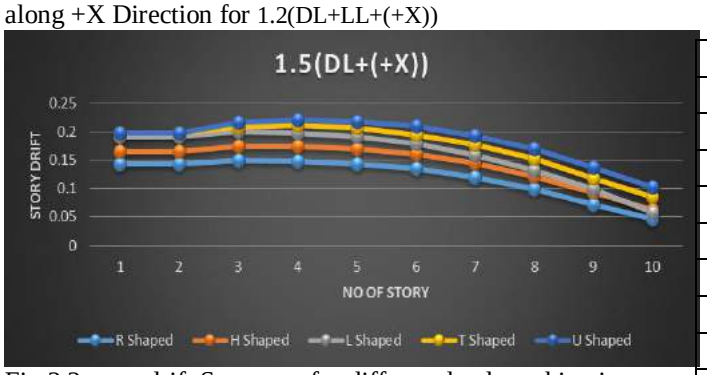


Fig 2.2 story drift Summery for different load combination along +X Direction for 1.5(DL+(+X))

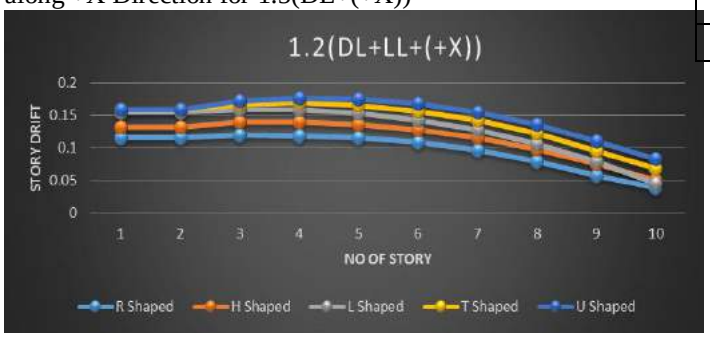


Fig. 2.2 story drift Summery for different load combination along +X Direction for 1.2(DL+LL+(+Z))

Table 4.1 Variation of Max B.M. and Shear Force of Column

Force	R-shape	L-shape	H-shape	U-shape	T-shape
Axial Forc Fx	3341.86	3624.262	3517.968	3634.199	3488.105
Shear Force Fy	214.572	262.411	240.258	260.328	264.961
Shear Force Fz	139.028	183.288	171.319	188.36	193.406
B.M. M_y	1970.004	2604.241	2432.286	2675.115	2747.06
B.M. M_z	2887.852	3845.152	3340.503	3651.996	3787.724

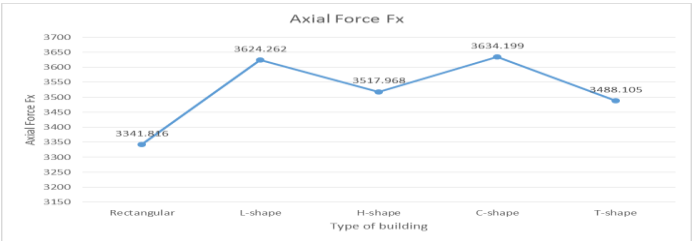


Fig 3.1Variation of Axial Forces for different shaped building.

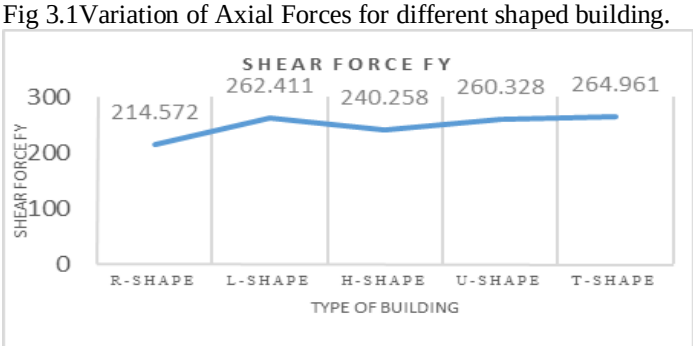
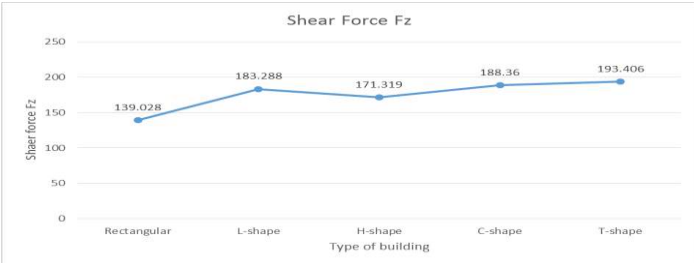


Fig 3.2Variation of shear force (Fy) for different shaped building.

Storey drift for load combination 1.2(DL+LL+(+X))					
	R Shaped	H Shaped	L Shaped	T Shaped	U Shaped
1	0.204	0.238	0.27	0.273	0.268
2	0.204	0.238	0.27	0.273	0.268
3	0.208	0.246	0.278	0.279	0.28
4	0.206	0.247	0.279	0.277	0.285
5	0.2	0.242	0.273	0.269	0.281
6	0.188	0.231	0.258	0.253	0.269
7	0.167	0.21	0.234	0.228	0.248
8	0.14	0.181	0.199	0.192	0.216
9	0.101	0.145	0.154	0.147	0.174
10	0.058	0.103	0.1	0.096	0.124



Variation of shear force (Fz) for different shaped building.

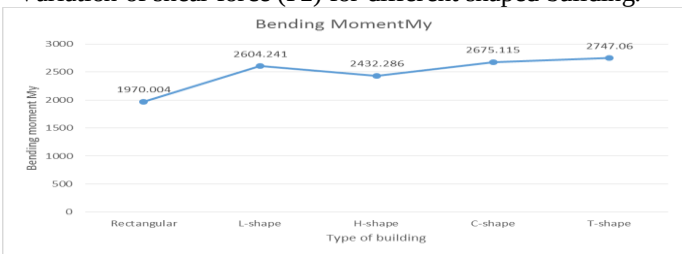


Fig 3.3 Variation of Bending Moment (My) for different shaped building.

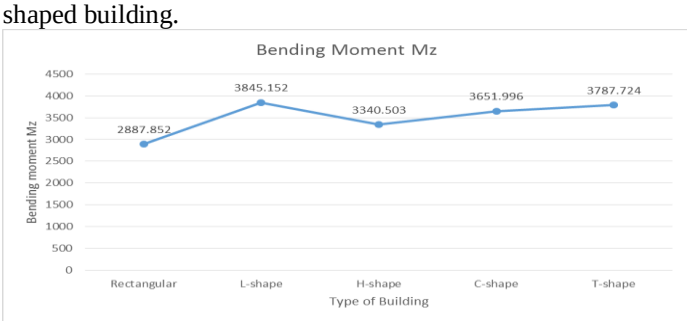


Fig 3.4 Variation of Bending Moment (Mz) for different shaped building.

Table 5.1Variation of Steel Required for Different Shaped Building.

Max area of steel required for different shaped buildings in mm2				
R- Shaped	L- shaped	U- shaped	T- shaped	H- shaped
7028	9709	9706	9824	8484

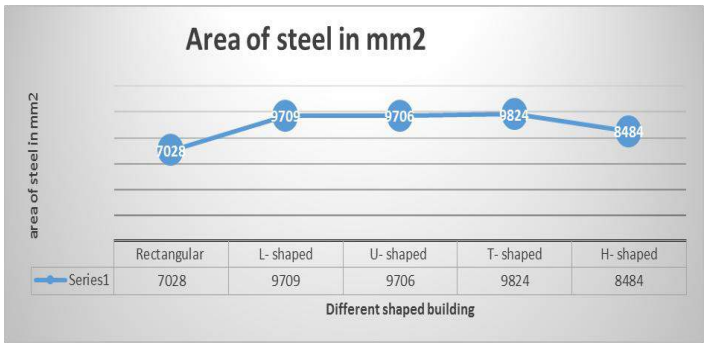


Fig 4.1Variation of Area of steel for different shaped building.

CONCLUSION AND REMARK:

1) It is proposed that buildingswith irregularities are prone to earthquakedamage, as observed in many earthquake occurrences.

2) Soft storey-For all new RC frame buildings,the best option is to avoid such sudden and large decrease in stiffness and/or strength in any storey; it would be ideal to build walls(either masonry or RC walls) in the ground storey also. Designers can avoid dangerous effects of flexible and weak ground storeys by ensuring that too many walls are not discontinued in the ground storey, i.e., the drop in stiffness and strength in the ground storey level is not abrupt due to the absence of infill walls. The existing open ground storey buildings need to be strengthened suitably so as to prevent them from collapsing during strong earthquake shaking.
The owners should seek the services of qualified structural engineers who are able to suggest appropriate solutions to increase seismic safety of these buildings.

3) Considering the effect of lateral displacement on different shapes of the building of the structure. it has been observed that, L-shape ,H-shape, T-shape and U-shape building have displaced more in both direction (X and Y) in comparison to other remaining simple shaped building.

4) The Area of steel is less for a regular building than irregular building. In the case of unsymmetrical building the amount of steel per unit area depends on the building irregularity.

5) irregular shape building undergo more deformation and hence regular shape building *must be preferred.*

6) Results have been proved that U shape building is more vulnerable compare to all other different shapes.

7) Considering all these above conclusions made on analysis of irregular structures, we may finally say that simple geometry attracts less force and perform well during the effect of earthquake.

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TECH-CHRONICLE
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DEVELOPMENT OF EPS BEADS BASED LIGHT WEIGHT GEOMATERIAL FOR WALL BUILDING UNITS

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***Abstract* – Nowadays, the demand of lightweight building materials has been growing worldwide. In this study Expanded Polystyrene (EPS) beads with fly ash are taken under consideration to manufacture the light weight bricks. It's a novel concept of using EPS beads as an alternative raw material for the production of fly ash bricks which are comparatively lighter in weight. The innovative application of high volume EPS beads in reducing the density of the composite material proved to improve the engineering properties of the geo-material.**

An extensive experimental investigation has been carried out to represent the varied engineering properties of the geo-material consisting of fly ash and EPS beads. There is reduction in unit weight and compressive strength with the addition of EPS beads, however, with the mixing of small percentage of admixture like cement the light weight geo-material can enhance the strength in addition to improve overall properties.

The average bulk unit weight of proposed light weight geo-material block obtained by using Fly ash, sand, cement and EPS beads is 1.25 gm/cc. After determining the block properties (water absorption, compressive strength, and unit weights), it was found that light weight geo-material block meets the requirements of the masonry standards used in India. These Blocks falls under the class 3.5 as per IS 12894:2002 having compressive strength not less than 35 Kgf/Sq.cm and water absorption not more than 20%. The obtained material is lighter than the commercial ones, which facilitates their rapid elaboration, quality control, and transportation.

Keywords: Fly ash, Expanded polystyrene, EPS, light weight blocks, unconfined compressive strength, water absorption, unit weights.

I. INTRODUCTION

In India Bricks are widely used in building construction as the most common building materials. The heavy weight of bricks accounts for the great mass of construction and thus causes more vulnerability against settlement and earthquake forces. In the present work, it is, therefore, tried to reduce the density of the bricks. Clay bricks are considered one of the most important building materials used to construct walls for buildings. Due to the unsustainable mining of clay soil for clay brick making, cement bricks have been introduced into the industry providing more alternatives. However, the production of cement bricks consumes an enormous amount of cement. Besides, the production of cement is not environmentally friendly. The manufacturing of cement is not only a high energy consuming process, but the production of each tonne of cement releases approximately 1 ton of carbon dioxide (CO2) into the environment due to the calcinations of the raw materials and the combustion of fuels. In light of the economic benefits, conservation of natural resources, energy saving and environmental friendliness, the use of alternative materials from waste products has become the main focus of engineers and researchers. This project aims at producing lightweight bricks by using the expandable polystyrene. Polystyrene is chosen due to its lightweight properties, with good energy absorbing characteristic and good thermal insulator.

Recently with rapid development of high rise building, floating marine platforms, large sized and long span concrete structure, light weight concrete(LWC) have been promising modern construction material. In comparison with ordinary clay bricks, light weight bricks shows excellent characteristics such as lower density, higher specific strength, better thermal insulation and greater energy absorption which can be obtained by replacing standard aggregate totally or partially by light weight aggregate (LWA). Light weight aggregates are broadly classified into two types: natural (pumice, diatomite, volcanic cinders) and artificial (sintered fly ash, expanded shale etc.). Expanded Polystyrene (EPS) is a type of artificial light weight aggregate with the density of only 10-30 kg/m^3.

II. MAIN AIMS AND OBJECTIVES OF THE PROPOSED WORK

The primary aim of this study was to investigate the feasibility of using a significant portion of fly-ash and EPS beads for beneficial purpose in civil engineering applications that is cost effective and environmentally friendly. The detailed laboratory investigations were planned and carried out for the determination of the best production method and the best mix design.

Thus, the main objective of the study undertaken may be summarized as to evaluate various physical parameters of the fly-ash and EPS mixes such as compressive strength, Water absorption, Dry and Wet Bulk unit weights and to select the optimal mixture of EPS among experiments under consideration to manufacture the light weight EPS blocks.

III. CRITICAL REVIEW OF LITERATURE

While going through various relevant literatures, various important facts about EPS beads are realized. Most of the available literature presents data on the use of pre-puff EPS beads mixed with dredged soil at higher moisture contents. There is lack of information about the behavior of EPS-fly ash mixes at optimum moisture content. This study, therefore, seeks to fill this gap.

Expanded polystyrene (EPS) is a polymeric (plastic) foam that in its generic appearance is white in colour. While the use of EPS as a packing material is quite well known, geotechnical specialist now recognize EPS as the most commonly used geofoam material, a type of cellular geosynthetic which has been in consistent use in geotechnical application since the early 1960s. There are different types of plastic foams which are interchangeably referred as geofoam. They include expanded polystyrene, extruded polystyrene, preformed sheets of polyethylene and foamed-in-place polyurethane. The ensuing discussion deals with expanded polystyrene (EPS) geofoam beads.

In 1972, the Norwegian Road Research Organization (NRRL) use EPS blocks as an ultra-lightweight fill on NR 159 road at Flom, near Oslo to minimize the effect of subsidence as a result of marshy soft ground. The road was built over a peat bog where the in-situ soil consists of 3 m thick layer of peat above 10 m of soft marine clay. The settlement rate due to the weight of the road construction in the previous years was of the order of 100 mm a year and reached 300 mm in 1972-73, accumulating to a total road surface drop of 600 mm below bridge level. An EPS layer of 1100 mm thick was laid by excavating the same portion of soil and further increasing the road level by providing a 500 mm thick pavement on the top the EPS layers. During the following 12 years the subsidence was 80 mm and then virtually no settlement occurred (Refsdal, 1985). This is the first application of the EPS geofoam as lightweight fill over soft ground and ever since its use has increased in all parts of the world.

One possible limitation of using EPS geofoam as lightweight fill is the buoyancy forces. Due to its very low density, ground water fluctuations will influence the geofoam causing it to easily float away if not properly secured with suitable surcharge or ground anchors by considering the maximum water levels in the area in calculations (Horvath, 1995; Frydenlund and Aaboe, 1996).

Other geotechnical applications such as thermal insulation above clay liners in landfills and waste-containment facilities, or adjacent to below-ground walls can be performed by using thinner sections of geofoam. In these applications, geofoam panels of 25 - 100 mm thick are usually needed. It is also possible to cut a block into intricate shapes by using hot-wire to be used as a casting form for architechtural and ornamental purposes (Horvath, 1994). Another EPS product employing pre-puff was developed specifically for the landfill application to avoid problems due to freeze and thaw (Benson et al., 1995).

EPS is also used in lightweight concrete owing to its lightweight, excellent heat preservation and sound insulation properties (Ravindrarajah and Truck, 1994; Miled et al., 2004; Babu et al., 2005). Waste EPS granules have been used in with horticulture soils to improve a number of soil characteristics like improving drainage, lightening heavy soils and improving water uptake capacity of soils (Scheirs, 1998). Furthermore, it was also used as sub-base materials for pavements and railway track beds (Hanna, 1978; Duskov, 1996; Miki, 1996; Beinbrech, 1996; Siderius, 1998), construction materials for floating marine structures and fenders in offshore oil platforms (Bagon and Frondistous-Yannas, 1976), sea beds and sea fences; as an energy absorbing material for buried military structures, (Cook, 1983; Perry et al., 1991) and tunnel covering (Beinbrrench, 1996).

IV. MATERIALS

A new alternative light weight building material was prepared by using EPS beads, fly ash, cement, sand and water. The EPS beads used for the preparation of light weight building material were spherical in shape having diameter in the range 2-4 mm. These highly compressible EPS beads had density $0.20 kN/m^3$. The fly ash was collected in dry state from Koradi Thermal Power Plant, Koradi, Nagpur, India. The percentage of basic chemical compounds present in the fly ash were SiO2 (63.52%), Al2O3 (26.89%), Fe2O3 (5%), CaO (1.23%). According to ASTM C618 the fly ash is classified as Class F. the ordinary Portland cement of 43 grade was used as a binding material. Potable water was used to mix these materials.

FLY ASH EPS BEADS CEMENT

V. EXPERIMENTAL PROGRAM

The experimental program was planned with an objective to understand and investigate the suitability of fly ash-EPS mix as a building material. The experimental program is divided in to two stages. In the first stage the physical properties of the presently available building material were investigated in the laboratory. In the second stage, laboratory model experiments carried out for the determination of the best mix design so as to select the optimal mixture of EPS among experiments under consideration to manufacture the light weight EPS blocks. The following chapter discusses the laboratory equipment and techniques utilized throughout the testing program.

A. Mix proportion

The mix ratio is defined as the ratio of two materials by weight. A pilot project work was also conducted before deciding the range of limits of different mix ratios. There is no consistent mix proportion adopted for all the cases. In all the mixes, the aim was to reach the target unconfined compressive strength after 28 days, after mixing. However, it is noted that the UCS is obviously influenced by the moulding water content. The work plan comprise of Mix proportions and preparation of specimens with several different combination of EPS beads, Flyash and Cement at suitable W.C.(%).

B. Specimen Preparation

The use of EPS beads in fly ash to produce lightweight building materials is a new concept. There are relatively few publications available on this topic. Where higher strength is required, stabilizing materials such as cement or lime is used. The fly ash, EPS beads, cement and sand was mixed with the mix ratios and dry mixing was carried out first. For a compound mix potable water was added slowly with mix ratio of (W/ FA+sand) and the mixing was carried out by means of hand to form homogeneous slurry. The EPS beads were then added to this slurry with the mix ratio of (B/ FA+sand). Finally this compound mix was poured into the mould.

C. Experimental Test

Experimental test for investigation of

- Compressive strength.
- Water absorption.
- Bulk unit weights.
  - When Dry
  - When Soaked

VI. DEVELOPMENT OF AN ALTERNATIVE LIGTWEIGHT BUILDING MATERIAL

A series of laboratory experiments were conducted to know the behavior of proposed light weight material made up of EPS beads, fly ash, cement and sand. The effects of different mix ratios on density, compressive strength, stress-strain behavior and water absorption of such material have been studied.

Laboratory model experiments carried out for the determination of the best mix design so as to select the optimal mixture of EPS among experiments under consideration to manufacture the light weight building material. In the experimental study, three different mix ratios were used to prepare the light weight building material. The mix ratio is defined as the ratio of two materials by weight. These ratios were EPS beads to fly ash component (B / FA+sand), cement to fly ash component (C / FA+sand) and water to fly ash component (W / FA+sand).

Several trial specimens of cylindrical shape and size Diameter = 3.8 cm, Height = 7.6 cm, C/S Area = 11.34 cm^2 were prepared in the laboratory with different mix ratios before deciding the range of limits of different mix ratios in the pilot project.

The pilot tests helps us deciding the determination of the best mix design so as to select the optimal mixture of EPS among experiments under consideration to manufacture the light weight building material. Based on the trial test results the specimens with new mix proportions of EPS beads, Fly ash, Sand and Cement at suitable W.C.(%) are prepared and kept for 14 days and 28 days curing. It is also decided to reduce the water content between 10 and 30 % to obtain better compaction.

VII. STUDY ON BLOCK SAMPLES OF SELECTED MIXES

The fly ash, cement and sand was mixed with the mix ratio of (C / FA+sand) and dry mixing was carried out first to form a uniform mix. For the compound mix potable water was added slowly with mix ratio of (W / FA+sand) and the mixing was carried out by means of hand to form a homogeneous slurry. The EPS beads were then added to this slurry with the mix ratio of (B / FA+sand), and mixing was continued till compound mix of these four material was formed. Finally this compound mix was poured into the specially prepared cuboid shape moulds having internal dimensions of 230 mm x 110 mm x 70 mm. After setting time of one day all the test samples were removed from the mould, covered with gunny bags and kept for curing period of 28 days in the water tank.

The light weight EPS fly ash blocks are prepared by hand compaction and are kept for the curing period of 28 days.

After removing from water tank blocks these blocks are tested in the laboratory for their compressive strength in soaked and dry condition under compression testing machine (CTM).

Table 2 Results of the laboratory test on light weight building blocks

Mix No.	S. ID	FA %	Sand %	Cement % (FA + Sand)	EPS % (FA + Sand)	Water % (FA + Sand)	Avg Comp. Strength Kg/cm^2 (Soaked)	Avg Comp. Strength Kg/cm^2 (Dry)	Avg Water Absorption %	Avg Bulk Density Dry Gm/Cc	Avg Bulk Density Soaked Gm/Cc
1	A1	70	30	12	0.5	25	32.2		17.75	1.157	1.35
	A2	70	30	12	0.5	25		35.98	18.4	1.152	1.35
2	B1	60	40	12	0.5	20	**38.31**		15.02	1.18	1.36
	B2	60	40	12	0.5	20		**42.46**	14.85	1.18	1.36
3	C1	60	40	12	0.6	25	27.23		13.55	1.13	1.29
	C2	60	40	12	0.6	25		31.98	14.6	1.1	1.24
4	D1	60	40	10	0.5	20	23.1		17.6	1.15	1.36
	D2	60	40	10	0.5	20		27.5	18.76	1.158	1.37

Based on the compressive strength, among all the mixes light weight geo-material prepared with mix No.2 is recommended for making light weight wall building units showing best results. These blocks exhibit the average compressive strength of 42.46 kg/cm^2, water absorption 14.85 % and dry density 1.18 gm/cc. The compressive strength of the proposed light weight geomaterial building block is comparable with the presently available burnt clay bricks and percent water absorption of the proposed geomaterial is below the permissible limit. These blocks falls under the class 3.5 Bricks as per IS 12894:2002 having compressive strength in the range 35 Kgf/cm^2 to 50 Kgf/cm^2 and permissible water absorption not more than 20%.

VIII. SUMMARY AND MAJOR FINDING

Different densities of light weight building material are developed using fly ash, EPS beads, sand and cement and their engineering behaviour is reported. The engineering behaviour includes bulk unit weights (dry and soaked), water absorption and compressive strength of a new alternative lightweight building material. Twenty four variations were made in the mix ratios and the effect of these mix ratios on bulk unit weight, water absorption and compressive strength has been studied. The major finding of the experimental work is summarized below:

- The unit weight of the lightweight building material is decreased from 1.2 to 0.6 gm/cc when B/ (FA+sand) ratio is increased from 0.5 to 1 % for both FA/sand ratio 70:30 and 60:40 when W/ (FA+sand) ratio is 25 % for all C/ (FA+sand) ratios 6%, 10% and 12%. The variation between densities of the lightweight building material to B/ (FA+sand) ratio is found to be linear.
- There is no significant change in the unit weight were observed when W/ (FA+Sand) ratio is decreased from 35 % to 20% for all C/ (FA+sand) ratios. This indicates that the unit weight of the light weight building material is mainly depending upon the percentage of EPS beads to be added with respect to the fly ash. However, there is considerable increase in compressive strength with the decrease in the percent of water. On decreasing the percent of W/ (FA+Sand) from 35 % to 25 % the compressive strength 33.86 kg/cm^2 to 40.56 kg/cm^2 for B/ (FA+Sand) 0.5 %.
- The compressive strength of the light weight material is found to be increase with increase in the percent of sand for all the mix ratios. The compressive strength of the light weight material increased from 35.27 kg/cm^2 to 40.56 kg/cm^2 on increasing the sand percent from 30 % to 40 %.
- Duration of curing period has a considerable effect on the compressive strength of the light weight building material. For all the mix ratios the compressive strength increases linearly with the variation between 7 days compressive strength and 28 days compressive strength.

IX. CONCLUSIONS

Based on the results of the experiments done on proposed light weight geomaterial, the block properties of the proposed light weight wall building blocks are listed below:

Average Compressive strength = 42.46 kg/cm2

% Water absorption = 14.85 %

Dry unit weight = 1.18 gm/cc

Soaked unit weight = 1.36

Further the proposed light weight geomaterial building blocks have many advantages like-

- Light weight
- Economical
- Environmental friendly
- Saving of fertile land, pure water
- More compressive strength
- Use of wastage etc.

Excellent bonding between bricks and mortar have achieved without interlocking cavities possibly because fly ash and cement belongs to the family of pozzolona. The results shows that the lightweight bricks are more safe, economical, having higher strength and low water absorption compare to conventional burnt clay bricks.

X. SUGGESTIONS ON FUTURE TESTING AND APPLICATIONS

The data, results, and interpretations contained herein expand the existing knowledge of Fly ash and EPS beads and provide a foundation for their potential use in construction industry for making light weight building blocks. Based on the knowledge and experiences gained throughout this research program a more detailed discussion of potential applications and additional work on the fire resistant properties, thermal insulation properties, dimensional stability and durability needed to support the use of EPS beads for its applications in the construction industry.

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SEISMIC ANALYSIS OF MULTISTOREY FRAME BUILDING G+5 WITH FLOATING COLUMN

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Abstract— *In present scenario buildings with floating column is a typical feature in the modern multistory construction in urban India. Such features are highly undesirable in building built in seismically active areas. The measures, involving stiffness balance of the first storey and the storey above, are proposed to reduce the irregularity introduced by the floating columns. In this paper investigated the effect of a floating column under earthquake excitation for G+5 storey frame. The analysis is done by the use of SAP 2000. In this work also studies the variation of the both structures by applying the intensities of the past earthquakes i.e., applying the ground motions to the both structures, from that displacement time history values are compared. This study is to find whether the structure is safe or unsafe with floating column when built in seismically active areas and also to find floating column building is economical or uneconomical.*

Keywords— Floating column1,response spectrum2, time history3.

1. INTRODUCTION

Many urban multistory buildings in India today have open first storey as an unavoidable feature. This is primarily being adopted to accommodate parking or reception lobbies in the first storey. Whereas the total seismic base shear as experienced by a building during an earthquake is dependent on its natural period, the seismic force distribution is behavior of a building during earthquakes depends critically on its overall shape, size dependent on the distribution of stiffness and mass along the height.

The geometry, in addition to how the earthquake forces are carried to the ground. The earthquake forces developed at different floor levels in a building need to be brought down along the height to the ground by the shortest path; any deviation or discontinuity in this load transfer path results in poor performance of the building. Buildings with vertical setbacks (like the hotel buildings with a few storeys wider than the rest) cause a sudden jump in earthquake forces at the level of discontinuity. Buildings that have fewer columns or walls in a particular storey or with unusually tall storey tend to damage or collapse which is initiated in that storey. Many buildings with an open ground storey intended for parking collapsed or were severely damaged in Gujarat during the 2001 Bhuj earthquake. Buildings with columns that hang or float on beams at an intermediate storey and do not go all the way to the foundation, have discontinuities in the load transfer path.

1.1 Floating Column

A column is a vertical member in building which transfer load to its footing. Floating column is also vertical member it does not rest on footing but it turn it rest on a main beam which transfer to load to other main column of the building. The bellow figure shows three floating column (Two on first floor and One in second floor)

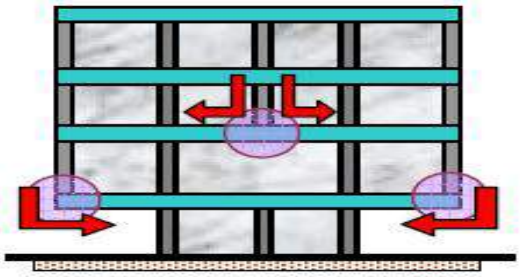


Fig 1.1 Floating column

There are many projects adopted in floating column, Mostly in above the ground floor because the more open space is available in the that floor. This column is considered in concentrated load our point load on a beam which is transfer the fully load on a beam. In the analysis of floating column

the column is assumed to the pinned at the base and therefore the reaction which it transfers to the beam is a point load. STAAD Pro and SAP2000 are tools required for floating column analysis.

1.2 The specific objectives of this research are

The objective of the present work is to study the behavior of multistory buildings frame with floating columns under earthquake excitations.
Building frame is subjected to real earthquake loading linear Time History analysis is carried out.
To study Linear seismic behavior of building frame with floating column Response Spectrum method.

Assumption regarding the base of the building frame is made and assumed to be fixed.

1.3 Outline of the Project Report

- Chapter 1 deals with introduction of present topic. It state brief about scope of present work.
- Chapter 2 throws light on the literature's review on multistory frame structure subjected to seismic analysis.
- Chapter 3 Various methods of seismic analysis.
- Chapter 4 studying the behavior of floating column for different soil condition under earthquake excitation and determining magnification factor for safe and economical design for building consisting floating column.

2. REVIEW OF LITERATURE

Reinhorn A. M., Simeonov V. K. and Mettupalayam S.V. (2000)

Presented an alternative approach to the formulation and solution of frame structures involving inelastic-nonlinear distributed-parameter structural systems. The response of the structure, which is spatially discretized, is completely characterized by a set of state variables which represent global nodal displacements and velocities, and local element quantities such as forces and strains at selected sections used as the integration points. A nonlinear beam element based on force interpolation functions and a constitutive macro-model is developed and presented in this framework. The state-space formulation and the nonlinear bending element, in particular, are compared with benchmark solutions using commonly used approaches.

Mundada A. P. and Sawdatkar S. G. (2014)

This paper deals with the study of architectural drawing and the framing drawing of the building having floating columns. Existing residential building comprising of G+ 7 structures has been selected for carrying out the project work. In this paper we are dealing with the comparative study of seismic analysis of multi-storied building with and without floating columns. The equivalent static analysis is carried out on the entire project mathematical 3D model using the software STAAD Pro V8i and the comparison of these models are been presented.

Starossek U., Lohning T. and Schenk J. (2009)

An approach for the nonlinear analysis of plane reinforced concrete frames is presented. Both material and geometrical nonlinearities including large displacements and rotations are considered. The approach is a combination of the displacement method and the transfer matrix method. An extension to space frames demands a sophisticated upgrading of the cross-section module to six internal, the computation of the spatial displacement values, and a consideration of interaction between the spatial internal forces.

Sezerin R. (2010)

Studied the geometrical non-linear analysis of the prismatic plane frames was researched with the stiffness matrix method by using the stability functions. Assumed the axial forces acting on the members as zero, the system was solved linearly under the initially set external loads, and the member axial forces were determined. The operations were repeated for each iteration. At the end of the iteration, the determinant and the eigen values of the system stiffness matrix, the system displacements and the axial forces of the members were determined. The external loads were regularly increased at the beginning of each iteration by multiplying them with a load factor, or the operations were continued until the least Eigen value became zero.

Mundada A. P. and Sawdatkar S. G. (2014)

This paper deals with the study of architectural drawing and the framing drawing of the building having floating columns. Existing residential building comprising of G+ 7 structures has been selected for carrying out the project work. In this paper we are dealing with the comparative study of seismic analysis of multi-storied building with and without floating columns. The equivalent static analysis is carried out on the entire project mathematical 3D model using the software STAAD Pro V8i and the comparison of these models are been presented.

Conclusion on Literature Review

1. The above literature conclude the behavior of building frame with and without floating column for static load, free vibration and forced vibration condition.
2. Studied geometrical non-linear analysis of the prismatic plane frames was analyzed with the stiffness matrix method by using the stability functions

3. SEISMIC ANALYSIS METHODS

In this chapter detailed study of multistoried building have been done with various types of loads and co-efficient.

The load considered for the analysis of multistoried building are:

1. Dead load 2. Live load 3. Earthquake load

3.1 Linear Methods of Analysis

Analysis of any structure can be done by following two methods:

1. Equivalent Static method 2. Dynamic method . (Response Spectrum method)

3.1.1 Equivalent static method

In this method is simpler, conservative and cost-effective. In many design codes and standards such as IEEE, it is stated that this Equivalent Static method is applicable to structures, which can be represented by a simple model and has simple dynamic characteristics as that of a cantilever beam. This method gives detailed analysis of structure. The procedure for Equivalent Static method is described below.

3.1.2 Response spectrum method

IS 1893 (Part 1): 2002 has recommended the method of dynamic analysis in section 7.8 in case of

- Regular building (h>40 m for Zone IV,V) & (h>90 m for Zone II,III)
- Irregular building (h>12 m for Zone IV,V) & (h>40 m for Zone II,III)

The purpose of dynamic analysis is to get the design seismic forces, along with its distribution among the various floor of the building and to identify various lateral loads resisting elements which is similar to equivalent lateral force method.

The methods of dynamic analysis described in the code is valid only for regular type of building, which is almost symmetrical in plan and elevation about the axis having uniform distribution of lateral load resisting element. It is further assumed that all the masses are lumped at the storey level and only sway displacement is permitted at each storey.

3.2 Nonlinear Methods of Analysis

Analysis of any structure can be done by following two methods:

- Pushover Analysis
- Time History Analysis

3.2.1 Pushover Analysis

In general, linear procedures are applicable when the structure is expected to remain nearly elastic for the level of ground motion or when the design results in nearly uniform distribution of nonlinear response throughout the structure. As the performance objective of the structure implies greater inelastic demands, the uncertainty with linear procedures increases to a point that requires a high level of conservatism in demand assumptions and acceptability criteria to avoid unintended performance.

3.2.2 Time History Analysis

Nonlinear dynamic analysis utilizes the combination of ground motion records with a detailed structural model, therefore is capable of producing results with relatively low uncertainty. In nonlinear dynamic analysis, the detailed structural model subjected to a ground-motion record produces estimates of component deformations for each degree of freedom in the model and the modal responses are combined using schemes such as the square-root-sum-of-squares.

4. EFFECT OF FLOATING COLUMN

4.1 Objectives

1. To study the behavior of a floating column for different soil condition under earthquake excitation.
2. As there is no provision or the magnification factor in I.S. *1893 (Part 1)2002*, IS *875 (Part 1): 1987, IS 875 (Part 2): 1987*, hence to determine such factors for design of floating column in a building.

4.2 Example Building Frames

For the analysis purpose a models have been considered namely as:

Model : Six storied (G+5) Special Moment Resisting Frame. This is two bays, 6 storey model, following models have been considered for Case 1.

Model 4.2.1- Building in which there are usual columns.

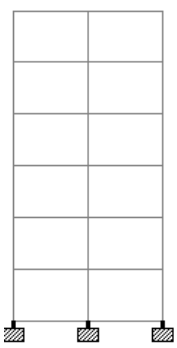
Model 4.2.2- Building in which there is floating column located at ground floor.

Model 4.2.3- Building in which there is floating column located at first floor.

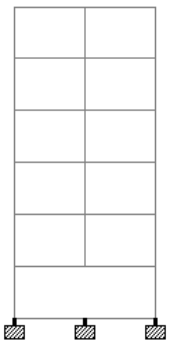
Model 4.2.4- Building in which there is floating column located at third floor.

Model 4.2.5- Building in which there is floating column located at fourth floor.

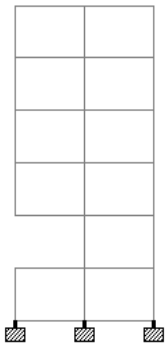
Model 4.2.6- Building in which there is floating column located at fifth floor.



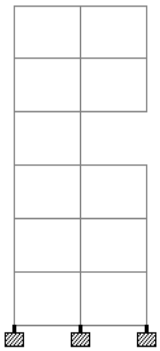
MODEL 4.2.1



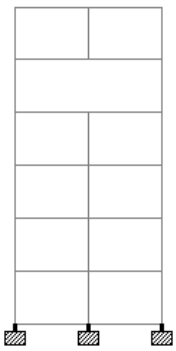
MODEL 4.2.2



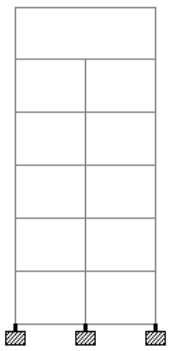
MODEL 4.2.3



MODEL 4.2.4



MODEL 4.2.5



MODEL 4.2.6

FIGURE 4.2 MODEL SIX STORIED (G+5) SPECIAL MOMENT RESISTING FRAME

Table 4.1 Details of Building Models

1	Type of Structure	Multi-storey rigid jointed plane frame (SMRF)
2	Seismic Zone	V
3	Number of stories	Six (G+5)
4	Floor to floor Height	3.5 m
5	Infill wall	230mm thick brick masonry wall
6	Type of soil	Medium and Hard
7	Size of column	350 mm x 400 mm
8	Size of Beam	300 mm x 450 mm
9	Live load	a) On roof = 1.5 KN/ m^2 b) On floor = 3.5 KN/ m^2
10	Material	M 20 Grade concrete & Fe 415 Reinforcement
11	Unit weights	a) Concrete = 25kN/Cum b) Masonry = 20kN/Cum
12	Total Height of Building	21 m for G+5
13	Damping in Structure	5%
14	Importance factor	1.0

4.3 Response Spectrum Analysis for Object 1

1) The variation in Base shear and Moments for various soil conditions to achieve the first object has been studied. RSA is done for the response spectrum corresponding to Zone V, hard soil and medium soil and 5% damping as per IS 1893 (2002), for all the frames. To represent the extreme cases floating column is provided at various floor level and at various positions.

2) The Base shears and Max. BM. From the RSA for hard soil and medium soil condition of the various model (4.2.2, 4.2.3, 4.2.4, 4.2.5 and 4.2.6) are given in Table 4.2. The percentage variations in each case are also listed (shown in parenthesis).

Table 4.2 Base shear and Max. B.M. on each floor

a) Zone V and Hard Soil Condition for 6 Storey Frame

Condition of floating column	Model 4.2.2	Model 4.2.3	Model 4.2.4	Model 4.2.5	Model 4.2.6
Base shear	451.16	383.08	401.28	446.58	452.5
Max. B.M. GF	530.01	337.95	350.4	388.35	394.2
1 Floor	488.62	412.57	324.58	345.81	351.6
2 Floor	293.54	386.79	302.6	287.09	310.8
3 Floor	260.8	347.44	344.25	359.64	251.6
4 Floor	197.5	293.87	289.47	228.22	256.4
5 Floor	131.5	211.62	207.17	185.82	99.44

b) **Zone V and Medium Soil Condition for 6 Storey Frame**

Condition of floating column	Model 4.2.2	Model 4.2.3	Model 4.2.4	Model 4.2.5	Model 4.2.6
1) Base shear	600.5 (25%)	489.41 (23%)	538.78 (25%)	590.11 (24%)	602.2 (25%)
Max. B.M. GF	707.3 (25%)	437.84 (23%)	470.96 (25%)	515.04 (24%)	526.2 (25%)
1 Floor	659.6 (25%)	560.57 (26%)	440.18 (26%)	465.61 (26%)	474.9 (26%)
2 Floor	399 (26%)	521.84 (26%)	411.53 (26%)	389.16 (26%)	422.6 (26%)
3 Floor	346.0 (25%)	459.97 (24%)	459.69 (25%)	473.85 (24%)	338.4 (26%
4 Floor	249.8 (21%)	382.58 (23%)	377.56 (23%)	287.42 (20%)	322.1 (20%)
5 Floor	131.5 (0%)	274.69 (23%)	268.22 (23%)	228.99 (19%)	118.6 (16%)

4.4 RSA for Object 2

The maximum magnification factors for base shear and bending moment has been evaluated to achieve the second object. Response Spectrum Analysis is done for the response spectrum corresponding to Zone V, medium soil and 5% damping as per IS 1893 (2002), for all the frames. To represent the extreme, cases floating column is provided at various floor level and at various positions. The Magnification factors from the RSA for the two building frames cases for various models are given in Table 4 & Table 6.

The base shear demands from RSA for both Case 1 (B-1) without floating column frames are found to be higher than that of other models having a floating column, in both the cases. The magnification factors in each case are also listed (shown in parenthesis)

Table 4.3 Magnification factors from RSA for 6 Storey Frame

a) shear and Maximum Moment in columns

Model		Model B-1	Model B-2	Model B-3	Model B-4	Model B-5	Model B-6
Base shear (kN)		611.7	600.5 (0.98)	489.4 (0.8)	538.8 (0.88)	590.1 (0.96)	602.1 (0.98)
Max. BM in GF (kNm)`	**IC**	534.1	0	394.3 (0.74)	471 (0.88)	515.0 (0.96)	526.2 (0.99)
	EC	462.1	707.3 (1.53)	437.8 (0.95)	410 (0.89)	446 (0.97)	455.1 (0.99)
Max. BM in 1stF (kNm)	**IC**	480.9	659.6 (1.37)	515.6 (1.07)	431.7 (0.9)	465.6 (0.97)	474.9 (0.96)
	EC	272	235.4 (0.87)	0	242.1 (0.89)	262.1 (0.96)	268.7 (0.99)
Max. BM in	**IC**	426.2	399 (0.94)	307.2 (0.72)	344.3 (0.81)	389.7 (0.91)	422.6 (0.99)

2nd F (kNm)	**EC**	250.9	246.5 (0.98)	295.9 (1.18)	329.1 (1.31)	252.5 (1.01)	246.8 (0.98)
Max. BM in 3rd Floor (kNm)	**IC**	358	349.0 (0.97)	269.1 (0.75)	400.6 (1.12)	473.8 (1.32)	338.4 (0.95)
	EC	216.2	207.1 (0.96)	272.5 (1.26)	0	146.8 (0.68)	217.3 (0.99)
Max. BM in 4th F (kNm)	**IC**	262.3	249.8 (0.95)	201.3 (0.77)	216.9 (0.83)	0	322.0 (1.23)
	EC	171	163.2 (0.95)	231.5 (1.35)	236.7 (1.38)	287.4 (1.68)	121.9 (0.71)
Max. BM in 5th F (kNm)	**IC**	137.4	129.7 (0.95)	112.5 (0.82)	127.4 (0.93)	229 (1.67)	0
	EC	79.5	131.6 (1.65)	113.2 (1.42)	234.6 (2.95)	124.1 (1.56)	118.5 (1.49)

b) Base shear and Maximum Moment in beams

Model		**Model 4.2.1**	**Model 4.2.2**	**Model 4.2.3**	**Model 4.2.4**	**Model 4.2.5**	**Model 4.2.6**
Max. BM in Beams (kNm)	**G F**	448.3	484.3 (1.08)	420.5 (0.94)	401.3 (0.9)	432.8 (0.97)	442.4 (0.99)
	1st F	466.9	450.5 (0.96)	560.6 (1.2)	440.2 (0.94)	451.3 (0.97)	461.3 (0.99)
	2nd F	401.7	387.5 (0.96)	521.8 (1.3)	411.5 (1.02)	388 (0.97)	395.6 (0.99)
	3rd F	307.9	294.7 (0.96)	460 (1.49)	459.7 (1.49)	337.7 (1.1)	302.8 (0.98)
	4th F	186.6	187 (1)	382.6 (2.05)	377.6 (2.02)	216.5 (1.16)	215.5 (1.15)
	5th F	79.48	131.5 (1.65)	274.7 (3.46)	268.2 (3.37)	124.1 (1.56)	118.6 (1.49)

4.5 Time History Analysis for Object 2

The maximum magnification factors for base shear and bending moment has been evaluated to achieve the second object. Time History Analysis is done for LOMA earthquake, for all the frames. To represent the extreme cases floating column is provided at various floor level and at various positions. The Magnification factors from the Response Spectrum Analysis for the two building frames cases for various models are given in Table 6 & Table 7.

The base shear demands from THA for 6 Storey Frame without floating column frames are found to be higher than that of other models having a floating column, in both the cases. The magnification factors in each case are also listed (shown in parenthesis)

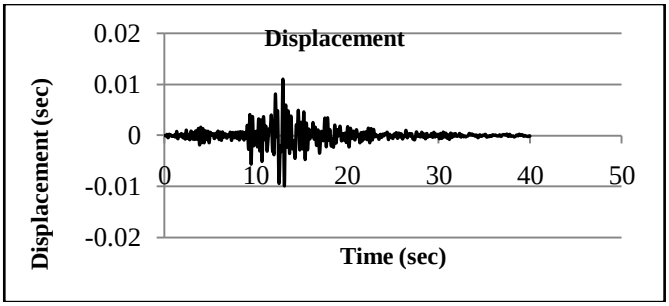


Figure 4.3 Displacement of top storey in model 1 for Loma Earthquake

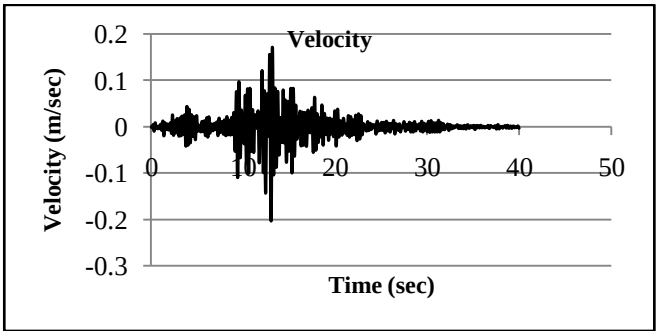


Figure 4.4 Velocity of top storey in model 1 for Loma Earthquake

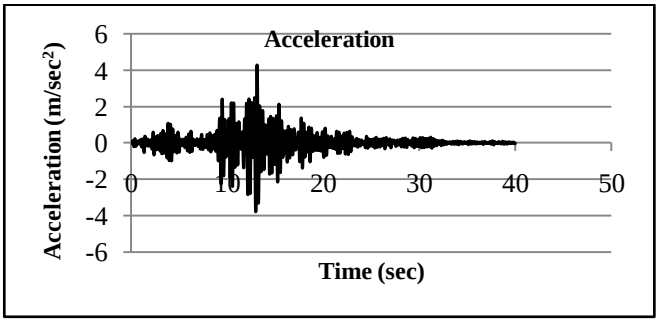


Figure 4.5 Acceleration of top storey in model 1 for Loma Earthquake

Table 4.4 Magnification factors from THA for Case 2

a) Base shear and Maximum Moment in columns

Model		**Model 4.2.1**	**Model 4.2.2**	**Model 4.2.3**	**Model 4.2.4**	**Model 4.2.5**	**Model 4.2.6**
Base shear kN		151.5	129.13 (0.85)	163.15 (1.08)	117.77 (0.78)	140.8 (0.93)	148.88 (0.98)
Max. BM GF kNm	**IC**	76.02	0	63.86 (0.84)	57.09 (0.75)	70.48 (0.93)	74.45 (0.98)
	EC	45.28	70.25 (1.55)	25.31 (0.56)	35.23 (0.78)	42 (0.93)	44.29 (0.98)
Max. BM	**IC**	98.25	96.55 (0.98)	111.1 (1.13)	74.56 (0.76)	92.33 (0.94)	96.42 (0.98)

1st F kNm	EC	57.13	44.87 (0.79)	0	41.69 (0.73)	52.83 (0.92)	56.06 (0.98)
Max. BM 2nd F kNm	IC	88.33	71.78 (0.81)	87.02 (0.99)	58.89 (0.67)	79.71 (0.90)	87.5 (0.99)
	EC	53.08	45.09 (0.85)	82.68 (1.56)	59.63 (1.12)	51.94 (0.98)	52.2 (0.98)
Max. BM 3rd F kNm	IC	70.52	58.76 (0.83)	71.22 (1.01)	60.39 (0.86)	89.5 (1.27)	68.13 (0.97)
	EC	43.41	35.86 (0.83)	71.32 (1.64)	0	27.87 (0.64)	44.03 (1.01)
Max. BM 4th F kNm	IC	47.72	39.46 (0.83)	47.29 (0.99)	39.61 (0.83)	0	57.39 (1.20)
	EC	30.95	25.65 (0.83)	55.08 (1.78)	13.66 (0.44)	53.21 (1.72)	23.59 (0.76)
Max. BM 5th F kNm	IC	24.11	20.02 (0.83)	24.36 (1.01)	23.22 (0.96)	33.64 (1.40)	0
	EC	14.57	12.08 (0.83)	52.45 (3.60)	10.6 (0.73)	14.37 (0.99)	22.45 (1.54)

b) Maximum Moment in Beams

Model		Model 4.2.2	Model 4.2.3	Model 4.2.4	Model 4.2.5	Model 4.2.6	Model 4.2.2
Max. BM in Beams (kNm)	G Floor	102.98	98.99 (0.96)	129.24 (1.26)	78.64 (0.76)	96.0 (0.93)	101.06 (0.98)
	1st Floor	103.76	86.47 (0.83)	169.06 (1.63)	83.96 (0.81)	97.8 (0.94)	102.21 (0.99)
	2nd Floor	86.88	72.44 (0.83)	144.06 (1.66)	59.63 (0.69)	82.66 (0.95)	86.06 (0.99)
	3rd Floor	63.22	52.31 (0.83)	116.07 (1.84)	10.97 (0.17)	71.05 (1.12)	62.55 (0.99)
	4th Floor	36.62	30.36 (0.83)	90.38 (2.47)	2.12 (0.06)	45.93 (1.250	41.98 (1.150
	5th Floor	14.56	12.08 (0.83)	52.45 (3.60)	10.61 (0.73)	14.37 (0.99)	22.45 (1.54)

4.6 Results and Discussions

In the present study, result of Response Spectrum Analysis for varying soil conditions has shown that the base shear demands for medium soil is found higher than that of the hard soil in this case. The variation of base shear for 6 storey Frame it is from 23% to 25%. The variation in moments is in the range of 16% to 26 % . The magnification factor in the case of G+5 has been found in the range of 0.8-0.98 for base shear, 0.71 to 2.95 for moments in interior and exterior columns and 0.9 – 3.6 for moments in beams.

5. CONCLUSIONS AND FUTURE SCOPE

In this study, the analysis of multistory with and without floating column buildings are done by SAP 2000 17 software using response spectrum analysis and time history analysis and we have got the following conclusion.

5.1.Conclusion for Object 1

The study presented in the chapter 4 investigated effect of floating column. The following conclusions were drawn based on the investigation.

1. The base shear demands for medium soil are found higher than that of the hard soil in both cases (G+ 6 model). As the height of the building increases, variation in base shear from medium to hard soil condition decreases. For different soil conditions (medium to hard) the max.
2. Moments varies from 16-26% for six storied building model. It has been found that max. Variation in values of max. Moments comes at the ground floor (26%) for both the cases whereas the min. variation comes at the top floor (22% for case 1 and 16% for case 2).
3. It can further been concluded that as the height of the building increases the variation of max. moments gets reduced for different soil conditions.

5.2.Conclusion for object 2

1. The Max. Magnification Factor for the moment based on linear analysis for G+5 is in the range of 0.94-1.65 for 4.2.2 model, 0.74-3.46 for 4.2.3 model, 0.81- 3.37 for 4.2.4 model, 0.9- 1.67 for 4.2.5 model, 0.71 – 1.49 for 4.2.6 model.
2. The Max. Magnification Factor for the moment based on nonlinear time history analysis for for G+5 is in the range of 0.79-1.55 for 4.2.2 model, 0.56-3.60 for 4.2.3 model, 0.67- 1.12 for 4.2.4 model, 0.64- 1.40 for 4.2.5 model, 0.76 – 1.54 for 4.2.6 model.
3. Results from Response Spectrum Analysis and Time History Analysis shows that the location of floating column at corners as in the model 4.2.3 and 4.2.4 is more critical than others.

The final conclusion is that do not prefer to construct floating column buildings. With increase in dimensions of all members also it is getting more displacements than a normal buildings and also the cost for construction also increased. So avoid constructing floating column buildings

5.3.Scope of future work

i. Effect of irregularity of structure can be studied.
ii. The modeling needs to be improved by considering soil condition and foundation.
iii. Material non linear can be consider.
iv. STADD. Pro, E-TAB software can be used.

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TECH-CHRONICLE
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Review on:Implementation of Information Communication Technology in Construction Industry for Material Management

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Abstract— In construction industry material management is related to planning, procurement storing and providing the appropriate material of right quality, right quantity at a right place at right time. Material management is carried out in industries to achieve economy, to complete project within time, to reduce wastage, to achieve good quality, to use all the available resources, to minimize chances of delaying or stopping of activities in construction. Traditional approach toward material management on construction projects leads to material waste, cost of which is high, delay in project occurs, and errors occurs. If the construction material is managed properly then economy would gain. The project will be completed within time, with less losses of material. Material management contains mainly 4 process i.e. planning, procurement, logistics and inventory. There should have coordinatorily and well communicatory material management is very essential. So that the proper management of this single largest component can improve regularity of various performing activities and the economy of a project and help to ensure timely completion of project. One of the major problems in delaying construction projects results in poor material management. This paper contains review on ICT implementation for material management in construction industry. There is need of implementation of new technology for proper communication in material management.

Keywords - ICT, material management, construction industry

1. INTRODUCTION

In construction projects, materials contain high costs so, to minimize the investment material management is required. Generally cost of construction materials varies with time. Sometimes it happens that whenever there is a need of material for construction activity, there is unavailability of material in market or the material is available at a high cost. So, to maintain a stock considering investment and economy factor material management is essential.

1.1 Material management

Material management is related to planning, procurement storing and providing the appropriate material of right quality, right quantity at a right place at right time. Material management is carried out in industries to achieve economy, to complete project within time, to reduce wastage, to achieve good quality, to use all the available resources, to minimize chances of delaying or stopping of activities in construction. Material management is important to achieve clean & clear work and to reduce losses of materials. Material management is carried out to decide, when the order of the material should be given so that work may not stop.

1.2 Material Management Process

In construction industry following material management process is carried out. Site Engineer requisition form Store check (if not available) Purchase Manager List of vendors (Some are fixed) Order In construction industry, bar chart is prepared and according to that activities proceed. And bar chart is regularly updated. As per the bar chart and requirement of material site engineer sends the requisition form to the store keeper and according to that requisition form store keeper issues the material to the labors. If material is not available in the store then the message is conveyed to purchase manager. If purchase manager is not in organization, then message is conveyed to the builder. The builder has list of vendors and according to requirement quotation of materials are invited and order is placed. Sometimes the vendors are fixed for some materials. The vendors are fixed because of quality assurance, the builder pays money after some days, the vendor may also be relative or friend etc

2. ICT NEED FOR MATERIAL MANAGEMENT

For every construction industry, material is required. In construction projects construction materials consume major portion of cost. Generally the cost of construction materials consist of 50% to 60% total cost of the project. In construction

industry material management is carried out to minimize wastage of material, shortage of material, damage of material, lack of storage space and delay in supply. The problems in material management will be never ending; it is possible that ICT implementation may be answer to overcome the challenge of materials management of in the construction industry. The dilemmas faced in material management can be overcome by adopting ICT –enabled solution that can help to support the effective material management of material activities. There should be centralized material management system .The documentation should be proper the tracking of material should be regularly done ,computerized system need to be used and use of modern material management technology needs to be done.

3. REVIEW OF PREVIOUS STUDIES

Shailesh Jayaprakash Pagar, R. V. Devalkar, M.C.Aher (2015) have worked in that area.They have discussed that Small and medium sized constructions present at large part of the construction sector in India .They have taken questionnaire survey and case study to describe how SCMs can improve their performance in material management, to reduce their cost and to improve the project quality through ERP system. In survey they have found that in the construction productivity in India reveals the major causes of the lack in productivity boils down to the improper deposited materials, improper material handling, improper material application and improper material deliveries .The problems in material management will be never ending, it is possible that ICT implementation may be answer to overcome the challenge of materials management of in the construction industry. The dilemmas faced in material management can be overcome by adopting ICT –enabled solution that can help to support the effective material management of material activities .They have found that there should be centralized material management system .The documentation should be proper the tracking of material should be regularly done, computerized system need to be used and use of modern material management technology needs to be done

Fara Diva Mustapa, Muzani Mustapa, Mohd Saidin Misnan, Syamsul Hendra Mahmud (2012) have worked in the area of material management. They have discussed, material shortage, delay in supply, price fluctuation, damage & wastage, lack of storage space of materials problems are which can be overcome with the use of ICT in material management. They have surveyed 10 construction firms those are working for more than 10 years in Sarwak at the different process of material management as well as to determine the requirements needed to increase the usage of ICT in material management. They found that, at the planning and procurement stage more ICT tools are used than the logistic and inventory. They found that modern technology like RFID and bar code has not been utilized and considered as nonexistent in construction firms for material management because of investment of high cost .They have found that the utilization of ICT for material management is considered to involve high hardware investment .Especially when other processes such as material logistics and handling requires a greater investment in ICT tools such as bar coding for tracking of materials. Besides the requirements for staff training and qualified ICT specialist on specific software knowledge is needed that makes the process more expensive.

Javad Majrouhi Sardroud (2012) research shows that construction materials and its components constitute more than 50% of total project cost. So, planning and managing materials is very important because it affects the cost and time of project. Generally material management is carried out on traditional method which is error prone and unreliable. ICT can provide timely and accurate information of materials to the project manager. RFID is a wireless sensor technology, based on the detection of electromagnetic signals and radio frequencies, which are used to capture and transmit data from or tag, so RFID identifies greatest technology in 21st century. Three broad categories of materials in a construction project are: bulk materials, engineered materials, and fabricated materials. The first category requires relatively short times for delivery after an order is placed, while the second and third categories require detailed drawings and samples so it requires several months. He has mentioned construction martial management problems. Safety, cost, accuracy, network, flexibility and scalability, ease of use, ambient environment, ruggedness, time these are the factors should be considered before adopting any new technology. He has discussed the working process of RFID and information of its components. He has mentioned the advantages and disadvantages of RFID and Bar code and how the RFID is effective than bar code technique

Narimah Kasim, Rozlin Zainal, Alina Shamsuddin, Naadira Che Kamarudin (2012) have discussed that the poor material management can affect the overall construction time, quality and budget. There Paper-based reports are mostly used to record and exchange information related to the materials component within a supply chain, which is problematic, error-prone, and inefficient. They have discussed that generally, emerging technologies such as wireless system, bar-coding and RFID are not being adequately used to overcome human error and are not well integrated with project management systems to make the tracking and management of materials easier and faster. Thus, the study seeks to identify the potential employment of that technology focusing on RFID for materials management in construction projects. They have discussed the materials management on construction projects and potential to employ RFID in materials management practices. For large projects material management, complexity always increases. According to them the ICT can give good facility for these large projects.

Khyomesh V. Patel, Prof. Chetna M. Vyas (2011) have discussed the objectives and functions of material management. They discussed the process as planning, purchasing receiving, inspection, stacking and storage, issuing material in material management problems at these processes different phases like material identification, vendor selection, procurement, and construction phase. They have taken survey of material management in Ahmadabad of three known builders. They found that, there should be centralized material management team co-ordination between the site and the

organization. In construction industry, proper control, tracking and monitoring of the system is required and also awareness & accountability should be created within the organization. They found that there is need of an efficient MIS integrating all aspects of material management.

Narimah Kasim (2011) worked in that area. He has discussed that ICT based technologies are emerging such as wireless communication, bar-coding and Radio Frequency Identification (RFID) for tagging technologies. Thus, an appropriate implementation of ICT could facilitate more effective and productive materials management processes. He has discussed that, in regard to the adoption ofICT for building project management in the Indian construction industry, mobile internet had been used only by 15% of the surveyed organizations for project management. It was even more apparent that RFID and bar-coding had not been used by any of the surveyed organisations for materials management. He found out the problems in material management such as lack of site storage spaces, small unloading area. ICT can reduce the level of confusion regarding the materials delivery from suppliers. He had taken a survey in Malaysia of Class A contractors (the highest contractor classification in Malaysia with financial limit RM 4 million and above), attempted to implement ICT, and the willingness of individuals within the organizations to participate and discussed their results. The main reasons of resistance towards the increased level of implementing ICT in materials management are due to the high cost of investment

N. B. Kasim, Peniel Ang Soon Ern (2010) have categorized material management to 5 processes namely planning, procurement ,logistics ,handling ,stock and waste control. They have taken interview and questionnaire survey of A class contractors in Malaysia which is highest contractor classification in Malaysia. The questionnaire survey was taken on implementation of ICT and interview was taken on acceptance of ICT for material management. They found that main barrier of implementation of ICT is high cost and there was just average level of acceptance of ICT by the industries. In construction industries, for material management, Microsoft office and handheld devices are widely adopted but bar code and RFID tools are not adopted. On the other hand, the main reasons of resistance towards the increased level of implementing ICT in materials management are due to the high cost involvement whether in the maintenance or overall implementation. In addition, the exceptional high cost of specialist software is also a barrier to the upgrading of ICT implementation.

Weisheng Lu, George Q. Huang, Heng Li (2010) Radio frequency identification technology has been applied in various areas such as retail, electronic transaction, manufacturing and assembly e.g. the assembly of cars, scientific research e.g. tracing snakes and migratory birds, security e.g. access control, medicine e.g. identifying a specific patient, express service e.g. American express etc. By adopting this technology real time information visibility and traceability has been increased but still RFID in the construction industry has not taken place. One possible reason is that construction practitioners may have not been fully informed of its potentials. The management arises in the construction because of complexity of modern construction projects and demand of speedy and efficiency of construction. For decision making, project manager requires real time information of materials, men and machinery such as inventory of materials, position of construction workers, condition of machineries etc. Main objective of information management is to make sure that accurate information is always available at the right time in the right format to the right person to support decision making. At other industries, at the supply chain management RFID increases the real time traceability, efficiency and quality eg. distribution of wholesale and retail. Construction industry is also information based so attention should be given for implementing ICT. RFID system contains RFID tag, RFID scanner and data base. RFID tag is nothing but micro chip at which data is stored. They have discussed about difference in bar code and RFID technology. RFID has been adopted in various areas such as Walmart. In construction RFID can be implemented for Management of materials (such as Logistic and supply chain management, Inventory management, Quality assurance, Waste management) for management of men (such as Access control and labour attendance record, Safety of men) for management of machinery (such as tracking of machines and tools, Machine operation and record, Machine maintenance record). For implementing RFID technique in construction hurdles should be considered such as technical, cost and health.

Vanita Ahuja, Jay Yang, Ravi Shankar (2008) Construction Company involves in one or more projects for a building project management, there should be combined effort and good communication between project manager and team. The communication problem can cause reworking, delay of project and building defects. Construction industries are lagging behind in using of ICT technique. For large construction industries, it is easy to adopt new technology but for SMEs it's difficult. In India most of the organizations are SMEs. The Indian construction industries have construction experience in all terrain and climates of every project and also have strengths in terms of manpower. The survey is carried in India and Perceived benefits were grouped under measures of project success, effective team management, effective use of technology and increased efficiency of the organization. Perceived importance of the identified benefits was measured at a 5 point Likert scale, with 1 and 5 corresponding to 'not important' and 'most important' respectively, whereas 3 corresponded to 'moderately important'. They have surveyed about status of adoption of ICT technique in construction industry. The construction industries which are having high turnover, they have adopted more number of ICT tools.

Table1: Methods Used For Ict Implementation

Shailesh Jayaprakash Pagar ,R. V. Devalkar, M.C.Aher	Questionnaire survey and case study to describe how Small and medium sized constructions (in India) can improve their performance in material management	Construction productivity in India reveals the major causes of the lack in productivity boils down to the improper deposited materials, improper material handling, improper material application and improper material deliveries.	The documentation should be proper the tracking of material should be regularly done, computerized system need to be used and use of modern material management technology needs to be done. ICT implementation may be answer to overcome the challenge of materials management of in the Indian construction industry.
Fara Diva Mustapa, Muzani Mustapa, Mohd Saidin Misnan, Syamsul Hendra Mahmud	Surveyed 10 construction firms those are working for more than 10 years in Sarwak (Malaysia) to determine the requirements needed to increase the usage of ICT in material management.	At the planning and procurement stage more ICT tools are used than the logistic and inventory. Modern technology like RFID and bar code has not been utilized and considered as nonexistent in construction firms for material management because of investment of high cost.	Utilization of ICT for material management is considered to involve high hardware investment .Especially when other processes such as material logistics and handling requires a greater investment in ICT tools such as bar coding for tracking of materials. Besides the requirements for staff training and qualified ICT specialist on specific software knowledge is needed that makes the process more expensive.
Javad Majrouhi Sardroud	Influence of RFID technology in material management	RFID effectively applicable for material tracking as every material have unique code.RFID systems are capable of reading multiple tags simultaneously and instantaneously.	Mentioned the advantages and disadvantages of RFID and Bar code and how the RFID is effective than bar code.Imporatnce of benefits of implementation of bar code and RFID technique for material management.
Narimah Kasim	A survey in Malaysia of Class A contractors (Malaysia)attempted to implement ICT	ICT based technologies are emerging such as wireless communication, bar-coding and Radio Frequency Identification (RFID) for tagging technologies.	As there are proactively much benefits of ICT techniques in material management still there is resistance towards the increased level of implementing ICT in materials management are due to the high cost of investment
Vanita Ahuja, Jay Yang, Ravi Shankar	Questionnaire survey is carried in India for ICT adaptation for building project management in construction industry	Perceived benefits were grouped under measures of project success, effective team management, effective use of technology and increased efficiency of the organization	ICT can increase speed of work, better communication and decreases documentation error.

4. CONCLUSION

ICT tools like Microsoft Offices and handheld devices are mostly used by construction companies for material management. Bar-coding and RFID are tools that are hardly adopted in the materials management of Construction Company.

Nevertheless, ICT transformation was deemed vital especially in the area of cost effectiveness and in materials handling. On the other hand, the main reasons of resistance towards the increased level of implementing ICT in materials management are due to the high cost involvement whether in the maintenance or overall implementation. In addition, the exceptional high cost of specialist software is also a barrier to the upgrading of ICT implementation.

India is a developing country but construction industries are lagging behind in technology use compared to the other industries.

In Indian construction companies there is no existence bar coding and RFID tools of ICT in material management. There should be a centralized material management team co-ordination between the site and the organization. Proper control, tracking and monitoring of the system is required. Awareness and accountability should be created within the organization. There should be use of new communication technology in the process of material management.

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TECH-CHRONICLE
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"Investigation of Economical Materials to Reduce Industrial Noise Level"

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1. ABSTRACT

Noise is 'unwanted sound' emitted from the vibrating body awareness of which to human being is by the physical sensation of hearing. Noise plays an important role in occurrence of annoyance, inconvenience and creating nuisance, which causes noise pollution affecting the quality of life. Noise pollution has become more prevalent in the present scenario and is still major ignored issue in industrial construction. Noise pollution is an environmental problem all over the world, which has very harmful effects on health and life of the workers in industrial sector. Workers are exposed to continuous noise throughout the workday, may leads to some injuries such as hearing loss (temporary or permanent), weakness in nerves, pain in internal tissues, heart problems, and even higher blood pressure in long term. It is seen from the experimentation that a long exposure to noise over 85 dB might be a dangerous factor for high blood pressure (BP), and it may induce major problems amongst the sensitive individuals and hence more focus is required on noise control. So we are trying to reduce ill effects of Industrial Noise pollution on Health of Workers by using Techno-Economically feasible Construction Materials in Industries.

This study emphasizes the vital role of Salvinia dust, *betung bamboo, Coconut Coir Fiber, recycled rubber particles etc* as natural sound reduction material, to give a solution for the existing industrial noise problems and also aimed to identifying the best practices in industries.

Keywords— *Noise Reduction Coefficient, Salvinia dust, Impedance tube, synthetic material, organic material, barrier and screen synthetic material, organic material*

2. INTRODUCTION

Effective noise control has become an important topic because of its various direct and indirect impacts of noise on the productivity as well as health of the worker. Traditional noise control techniques mainly include insulating, absorbing, vibration isolation and damping. However they have their own limitations. Presently, due to the technological advancement lot of new construction materials are available in the market to reduce the problem of noise. With the help of these materials the building can be constructed where noise can be controlled more effectively in economical way.

Presently, in the industrial construction the problem of undesirable and potentially hazardous noise has become much more complex and serious; the demands for a better environment and quality life styles are increased. However owners and architects are not paying much of the attention to control the noise pollution. Most of the developed countries use practical techniques to minimize the nuisance such as barrier walls, duct silencers, acoustical wall panel, sound proof curtains, sound enclosures for industrial machinery and other similar noise control treatments that are installed near the source to effectively reduce the sound level. However, India has not yet yielded much into this issue as noise reduction methods are costly. Therefore, it is necessary to find out cost effective solution to control industrial noise.

It is very difficult to estimate and control the noise at different workplaces in industry. However the motivation behind this project is the availability of lot of new sound

absorption material with different acoustical properties. The noise levels have to be estimated in the working condition and depending upon the working condition suitable type of construction material is to be applied so as to control the noise effectively in industrial sector. Consulting the importance of noise pollution and availability of new materials; in this dissertation it will be attempted to study the techno-economical feasibility of the various new construction materials for the effective noise control in industrial sector.

2.1 GENERAL

For Reducing ill effects of Industrial Noise pollution on Health of Workers by using Techno-Economically feasible Construction Materials in Industries we can use following materials.

There is an increasing importance of noise pollution in industrial sector because of its direct and indirect impacts on health of the workers and productivity. It is also seen from the literature review of manufacturer's catalogue that there are new construction techniques and materials which are developed in this decade due to the technological advancement. Hence in this dissertation the techno-economical aspects of the various construction techniques and materials for the effective noise pollution control will be studied.

2.2 OBJECTIVE OF PROJECT

1. The Scope and objectives of the dissertation are as given below:

a. To study the important sources of noise in press shop industry.
b. Study of new construction materials for noise reduction and their applications.
c. To study the effects of mix proportion, particle size and thickness of the product on effective noise reduction properties of the materials.
d. To study the different material combinations with allied artificial sound insulating material available in market.
e. Cost effectiveness of new construction materials and their suitability.
f. The above points will be studied with the help of case study in the form of live project and experimental work in the laboratory.

3. LITERATURE REVIEW

3.1 RIGID PERFORATED SCREEN

Kuo-Tsai Chen elucidates the calculation of sound transmission loss of a perforated screen at frequencies below 4000 Hz by using a two-dimensional plane wave theory and laboratory measurements from 125 to 4000 Hz, and the results of theory and experiment were compared. In the paper it has been clearly shown that the calculation of acoustic transmission loss through a perforated screen agrees well with measurements at frequencies above 315 Hz and the transmission loss depends on the thickness and the percentage of perforation. It also illustrated the calculation and measurement of the transmission loss below 4000 Hz through a perforated screen of different materials; thicknesses and percentages of perforation are included. The results show that the calculated transmission losses of all adopted perforated screens agrees well with those of the measurements, and the above transmission losses are largely independent of the chosen material of the screen if all dimensions are fixed.

This also highlight the technique of noise control using a sound absorber consisting of a perforated facing backed with a porous material, and is commonly used as a sound barrier, muffler in any of the practical acoustic application.

3.2 RECYCLED RUBBER PARTICLES

New kind of sound absorber using recycled rubber with its attractive characteristics; low-cost, broadband sound absorption, thin in thickness and relatively simple processing. This paper throws light on sound attenuation of recycled rubber particles and its influence on acoustic properties of porous material, perforated material and composites (foam, glass wool etc.) in layers or in double layers. Recycled rubber of density 1001.5 kg/m3, highly irregular shapes and sizes ranging from 150–840 µm have selected for Specimen preparation. Two-microphone impedance tube (type 4206) of Bruel & Kjaer has applied to measure the normal incident absorption coefficient and other acoustic parameters according to the standard procedure detailed in ISO (10534-2). The frequency range of measures from 100 to 1600 Hz. Form observation it has been seen that the high sound absorption in smaller size particles and low sound attenuation in larger size particles.

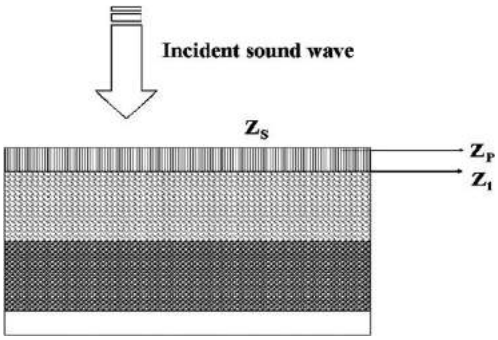


Fig. Generalization of acoustic transmission analysis diagram of composite absorber

The sound absorption ability of the absorber were greatly improved through impedance matching design of structure and combination of damping effect with conventional visco-thermal mechanism as well as resonating principle. Although more work is required to verify the effect of different rubber particles and composite structure, the results can lead to a novel kind of sound absorption materials with high performance.

3.3 COCONUT COIR FIBRE WITH PORUS LAYER BAKING AND PERFORATED PANEL

The Noise Absorption Coefficient (NAC) of coconut coir fiber was increased at all frequency when they were backed with Woven Cotton Cloth (WCC) and perforated panel with 0.20 perforation ratio tested to measure its sound absorption coefficient. At low frequency, the NAC have significant increased. This is because higher flow resistivity of WCC than coconut coir fibers; so that sound can be dissipated as it travels through material significantly. The results from the experimental tests showed that it has good acoustic properties at low and high frequencies and can be used an alternative replacement of synthetic based commercial product.

The experimental data indicated that porous layer backing improves noise absorption coefficient at low and high frequencies with significant increasing. 20 mm thick layer coconut coir fiber with porous layer backing exhibit peak value at frequencies between 2750-2825 Hz with maximum value of 0.97. The experimental results also found that the coconut coir fiber with perforated plate gives higher value for lower frequencies range from 600- 2400 Hz. The optimum value for coconut coir fiber with perforated panel is around 0.94-0.95 for the frequency range 2600-2700 Hz. By using the porous layer and perforated plate backing to coconut coir fiber, the sound absorber panel shows a good potential to be cheaper, lighter and environmentally friendly product compared to glass fiber and mineral based synthetic materials. The coconut coir fiber backing with woven cotton cloth was found better than tea-leaf fiber backing with woven cotton cloth. Hence in this dissertation here it is scope to work with coconut coir fibers with some more additives from natural, agricultural or artificial materials.

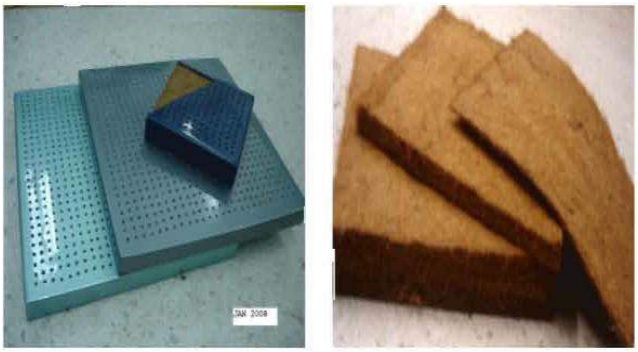


Fig Coconut Coir Fiber

3.4 SALVINIA DUST

In the Present paper author has shown the potential use of Salvinia dust as natural sound reduction material, to give a solution for the existing industrial noise problems. Specimens having a size of 75 mm (diameter) x 25 mm (thickness) were made by using Salvinia dust. Salvinia dust was mixed with cement at a ratio of 1:1 and water was added. Noise Reduction Coefficient of these specimens was investigated by using an experimental set-up consisted of speaker at one end, propagation tube and a noise level meter. Noise reduction ability of the materials was quantified by using Noise Reduction Co-efficient (NRC). Also it has been found that NRC increases with increasing the tile thickness. Also it was found that tile having grooved surface has shown much higher NRC values than the tile having flat surface.

Fig1-Salvinia moss

Fig 2- Specimen used for noise testing

From study it has been clearly shown that effect of specimen thickness on NRC of 50mm thick specimen is higher than that of 25 mm thick specimen in the frequency range 0.4 - 3.2 kHz. In the frequency range of 1.2 – 5.6 kHz the NRC for the 50 mm specimen varies between 0.2 - 0.4.Particle size effects on NRC is generally greater for the specimen cast with large particles than that for the specimen cast with small particles. Also the effect of mix proportion on the NRC of specimen having cement: Salvinia dust = 2: 1 mix ratio was higher than that of the other specimen, having less mix proportion, mainly the amount of cement. High cement ratio might contribute to increase flow resistivity, consequently causes to increase the NRC. In this Paper author has been concluded Salvinia dust is natural material, which can be effectively used to manufacture a product with appreciable noise reduction properties. However, durability and strength of the product not checked.

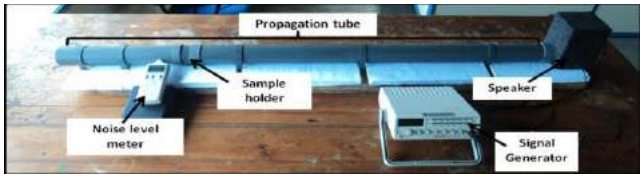


Fig3- Impedance tube system for Sound absorption measurement

3.5 RECENT TRENDS IN POROUS SOUND-ABSORBING MATERIALS

This paper deals with various sound absorber materials since 1960's in regards with their drawbacks and ineffectiveness and their ill effects on the workers while processing these materials. In the 1970s, sound-absorbing materials from asbestos-based materials to new synthetic fibers changed although these new fibers were not much safer because of issues related to human health and global warming led the use of natural fibers instead of synthetic ones.

Authors have highlighted the subsequent development in sound absorbent material compared with the older absorbing materials produced in the 1960s. The new materials have become safer, lighter and more

technologically optimized. In addition, the concept of environment friendly, sustainable, recycled, and green-building materials will soon have an important role in the marketing of sound-absorbing materials.

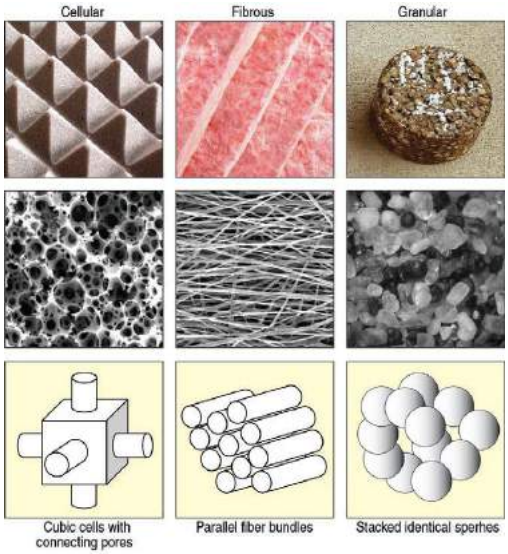


Figure . The three main types of porous absorbing materials.

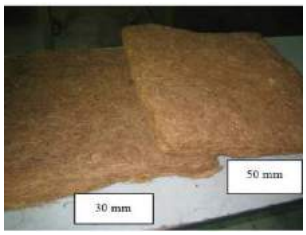
3.6 BETUNG BAMBOO

It has been observed from the results that sound TL and STC values of medium-density particleboard (0.8 g/cm3) were better than low-density (0.5 g/cm3) board. However, low-density particleboard performed well as sound absorber panels. Generally, the boards absorbed sound at low (< 500

Hz) and high frequency ranges (> 1000 Hz) and reflected sound at middle frequencies. Low-density boards (0.5 g/cm3) had a higher sound absorption coefficient than medium-density particleboards (0.8 g/cm3). Meanwhile, medium-density boards were effective as insulation boards, with higher TL and STC values when compared with low-density particleboard. Noise control technique of a sound absorber Particleboards made from Betung bamboo have promise for further development as acoustical construction material compared with other wood or natural composites as it has good Characteristics at the high frequency range

3.7IMPROVEMENT OF DATE PALM FIBRE ACOUSTIC PROPERTIES USING PERFORATED PLATE, WOVEN COTTON CLOTH AND POLYESTER

This paper throws light on effective use of date palm fiber as a sound absorber. The study has been carried out on the effect of date palm fiber as backing on sound absorption using three types of perforation plates and porous layers. The researchers has been used innovative material of date palm fibre as a backed with various 1-mm-thick zinc perforation ratios and hole diameters: 10% with 3 mm hole diameter, 10% with 4 mm whole diameter and 22% with 2 mm hole diameter.



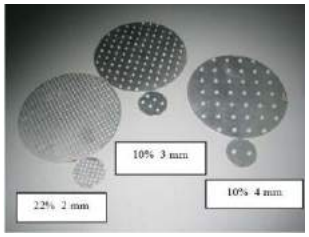


Fig.: sheet date palm fibre *fig perforated plate*

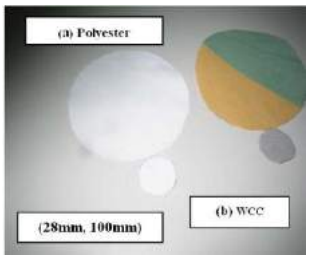


Fig Photographs of the test (a) woven cotton cloth (b) polyester

This Paper also shows that this innovative material has a promising future because it is much cheaper and lighter than industrial substances. Date palm fibre backing with a 10% perforated plate and 3 mm hole diameter exhibits a considerably higher acoustic absorption coefficient at low frequencies for 30-mm-thick backing and 22% with 2 mm hole diameter at all frequencies for 50-mm-thick backing. The empirical test results illustrated shows material has good acoustic absorption in the lower and higher frequencies and the date palm fibre backing with WCC performs best at low frequency and shifts to lower frequencies for both thicknesses. It has been concluded in this paper that the of perforated plate backing, woven cotton cloth and polyester with date palm fibre exhibits the high quality of acoustic absorber plates with easy production and environment friendliness application

4. CONCLUSION

For Reducing ill effects of Industrial Noise pollution on Health of Workers by using Techno-Economically feasible Construction Materials in Industries we can use above materials effectively.

Presently, in the industrial construction the problem of undesirable and potentially hazardous noise has become much more complex and serious; the demands for a better environment and quality life styles are increased. However owners and architects are not paying much of the attention to control the noise pollution. Most of the developed countries use practical techniques to minimize the nuisance such as barrier walls, duct silencers, acoustical wall panel, sound proof curtains, sound enclosures for industrial machinery and other similar noise control treatments that are installed near the source to effectively reduce the sound level. However, India has not yet yielded much into this issue as noise reduction methods are costly. Therefore, it is necessary to find out cost effective solution to control industrial noise.

It is very difficult to estimate and control the noise at different workplaces in industry. However the motivation behind this is the availability of lot of new sound absorption material with different acoustical properties. The noise levels have to be estimated in the working condition and depending upon the working condition suitable type of construction material is to be applied so as to control the noise effectively in industrial sector. Consulting the importance of noise pollution and availability of new materials; in this dissertation it will be attempted to study the techno-economical feasibility of the various new construction materials for the effective noise control in industrial sector.

From the literature review it is seen that the architects and engineers are not providing much of the attention to control the industrial noise which is otherwise very harmful to the health of the workers and directly affects the production. This may be mainly due to the cost of the materials which are commonly used to control the noise. It is also seen from the literature that the conventional materials and methods has their own limitations. From the literature study has also indicated that the new construction techniques and materials are available in the market and they can be used more effectively for efficient noise control

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