WSRR 13

24th and 25th September 2013

VMCC Main Auditorium
IIT Bombay
Mumbai-400076





Full Scale pushover test to validate performance of Composite FRP at CPRI Bangalore (2010)

Proceedings of the

WORKSHOP ON STRUCTURAL REHABILITATION AND RETROFITTING USING CONSTRUCTION CHEMICALS

ASTR
Association of Structural Rehabilitation

Indian Institute of Technology Bombay

FICCI
Federation of Indian Chambers of Commerce and Industry

Proceedings of the Workshop on Structural Rehabilitation and Retrofitting using Construction Chemicals – 2013 (WSRR 13)

VMCC Main Auditorium
Indian Institute of Technology Bombay
Mumbai – 400076

24th -25th September 2013

Editors
G. R. Reddy

Prakash Nanthagopalan

Pragyensh Kumar

Sanjay M. Takekar

Yogesh Singh Indolia

Association of Structural Rehabilitation

Indian Institute of Technology Bombay

Federation of Indian Chambers of Commerce and Industry

भारतीय प्रौद्योगिकी संस्थान मुंबई
पवई, मुंबई 400 076, भारत
Indian Institute of Technology Bombay
Powai, Mumbai-400 076, India

Office : 2572 3488, 2576 7001
Res. : 2572 3738, 2576 8000
EPABX : 2572 2545, Extn.: 7001 (O), 8000 (R)
Fax : 91-22-2572 3546 (Direct), 2572 3480
E-mail : director@iitb.ac.in
khakhar@iitb.ac.in
Website : www.iitb.ac.in

IIT Bombay

Devang V. Khakhar, Director
देवांग वि. खख्खर, निदेशक

Message

The Urban population is growing exponentially due to the multi-faceted job opportunities available in the cities. This demographic growth in and around urban areas demand huge infrastructural establishments. Considering the infrastructural needs, the Government of India has allocated approximately Rs. 50 lakh crores for the infrastructure development in India in this year budget (2012-13). However, several constructed facilities like buildings, pavements in the urban areas and also historic buildings and monuments are showing premature distress due to various environmental conditions and natural calamities. In this regard, rehabilitation and retrofitting becomes inevitable to restore the structures to the service level and increase their service life.

I am pleased to note that the Department of Civil Engineering in association with Association of Structural Rehabilitation (ASTR) and Federation of Indian Chambers of Commerce and Industry (FICCI) is organizing a Workshop on ***Structural Rehabilitation and Retrofitting Using Construction Chemicals 2013 (WSRR 13)*** during 24th – 25th September, 2013. It is envisaged that the Workshop will provide a comprehensive understanding about the process of investigation, repair, strengthening and rehabilitation of constructed facilities to the participants.

I congratulate the team for organizing this event and wish the Workshop a grand success.

[Devang Khakhar]
September 19, 2013

VIANY MATHUR
Deputy Secretary General

Federation of Indian Chambers of Commerce and Industry

Federation House
Tansen Marg
New Delhi 110001
T +91 11 23738760 (11 lines)
F +91 11 23320714 / 23721504
E ficci@ficci.com
www.ficci.com

MESSAGE

Construction Chemicals are an important component of chemical industry and are playing a major role in nation building. Their judicious usage improves the quality as also durability of structures. The use of Construction Chemicals is still negligible in India because of the low market awareness. With proper awareness about this sector the use of construction chemicals will increase and thus contribute in a small but healthy way to the Indian economy.

Federation of Indian Chambers of Commerce and Industry (FICCI) in association with Indian Institute of Technology, Mumbai and Association of Structural Rehabilitation (ASTR) are organizing a Workshop on Structural Rehabilitation and Retrofitting using Construction Chemicals. This workshop aims at creating a clear understanding among participants on the complete process of investigation; repair, strengthening and rehabilitation of reinforced concrete structures with appropriate application of construction chemicals. I am sure after attending the seminar, the participants should be better equipped to make right decisions on the selection of proper diagnosis and retrofit techniques as well as be able to effectively specify and execute/ supervise repair & strengthening works

I wish the Workshop a grand success.

Vinay Mathur

(Vinay Mathur)

Foreward

Structures will define the historical, economic and development status of the country. The structures could be heritage, residential, office, lifelines such as roads, rail, hospitals etc, Structures when exposed to environmental conditions such as sun rays, drying and wetting, pollution etc it looses aesthetic looks as well as strength. Stone structures especially heritage structures loosing its architectural values and some of them which are not well maintained are being collapsed due to many reasons. One of the important reasons could be loosing strength of foundation including supporting soil or rock. Coming to new structures which are mainly made of reinforced concrete or fully steel the situation is very surprising. There are large numbers of structures being collapsed every year under normal loads whose life may be in the range of 10-50 years. There may be stray incidents of newly built structures being collapsed. If the structures are exposed to natural events such as earthquakes, extreme winds, cyclones, tsunami etc., understanding their performances is not very simple. However, large number of scientists and engineers are putting efforts continuously to estimate the risk in these situations in the country. The risk could be in terms of economy and social aspects. The important parameters which contribute to risk are vulnerability of structures, type of hazard and exposure. Vulnerability definition is very specific to the type of hazard. For example, stone masonry will be highly vulnerable for earthquakes where as it may not be vulnerable to high winds. Vulnerability of RC structures varies from sea cost to interior regions of the country even under environmental conditions.

Considering the entire above facts, one should rehabilitate or retrofit the structures whenever there is a call. Otherwise, loss of life and economy will be very large which cannot be quantified. There is a technology available for this purpose. The difficulty is that one way the technology is not transferred to the needy in simple and adoptable way. Other way is that the public awareness in the technology is minimal. Sometimes it is assumed that the cost of repair or retrofitting is very expensive. To fill this gap regularly workshops are being organized and WSRR13 is one of them. In this workshop it is expected that research community, designer community, industrial/application community and administrative community will join and discuss various aspects of rehabilitation and retrofitting. It is also expected that large number of students from various institutions will actively participate in the workshop and take the subject ahead and try to put the nation in global picture.

To maintain and continue these activities non governmental, non profitable and social association called Association of Structural Rehabilitation (ASTR) has formed and trying its best to serve the public. There is large number of members and few local chapters and those are not members are requested to join us and serve the nation to make it resilient and stand top in the world.

With best wishes

G. R. Reddy
President
Association of Structural Rehabilitation (ASTR)

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CASES OF INVESTIGATION AND REMEDIAL SUGGESTION FOR CORROSION & CONSTRUCTION RELATED DISTRESS OF CONCRETE STRUCTURES

B. Bhattacharjee

Abstract: Concrete is not maintenance free and quite often it becomes necessary to undertake repair work of concrete structural element-during their service life. Besides, occasionally faulty construction practice leads to structural defects/distress in some structures necessitating repairs and rectification work. Such repair decisions are preceded by condition survey of structure by instrumental and non-instrumental means. In this paper, cases of in-situ testing for decision making for repair and rehabilitation of some structures are illustrated. Lastly the proposed remedial suggestions made are also highlighted Keywords: repair, in-situ testing, condition survey, distress, diagnosis.

INTRODUCTION

Concrete being a manmade material produced with expense of energy cannot be stable and would have a tendency to undergo changes due to increased chemical potential. The changes lead to deterioration and loss of functional performance resulting in repair and maintenance. Damages to concrete structure may occur due to overloading. Defects are due to faulty design, detailing and execution. Quite often it is not easy to foresee the implication of environment and improper quality control during concrete construction leads to distress due to deterioration such as rebar corrosion. Thus repair of structure becomes inevitable. Prior to such repair one has to estimate the degree of damage and extent of repair needed and for this purpose. In this article case study of such investigation is presented in the beginning. Followed by case investigation of one situation where by investigation had to be carried out on precast concrete elements which exhibited signs of various defects due poor curing practice adopted and poor quality control during casting. Lastly another case study related to improper execution is also presented.

CORROSION INVESTIGATION

The case building is nearly 25 years old. Due to ongoing expansion project, assessment of distress in the basement and two upper stories of the building were required. The foundation of the building was already designed envisaging the future expansion, in the meantime however, visible spalling of cover concrete and rusting of rebar became evident and hence the necessity of investigation. The objectives of the investigation were formulated accordingly as given below [1].

OBJECTIVES OF THE OVERALL INVESTIGATION

A quick look in to the buildings revealed significant visible corrosion distress at any locations. Hence the objectives of the overall investigation for the building are: (i) to assess the existing quality in terms of its uniformity and degree of variation, in the concrete. (ii) To check the aspects related aging namely carbonation and chloride content in the concrete. (iii) Degree of corrosion intensity and suggesting appropriate remedial measure.

VISUAL SURVEY AND CONDITION INDEX

First and foremost activity in a condition assessment and investigation is the visual survey. Every distressed element was surveyed and given a rating according to the codes proposed and methodology developed at IIT Delhi and available in literature [2,3]. The corresponding condition indices for every element were calculated using VICAS tool, developed at IIT [3]. According to this rating the worst with highest repair priority is assigned a value of 5 and lowest value of 1 stands for no repair requirement. Combined condition index for the whole building is also estimated using same tool and these condition indices of elements ranged up to 4.08. Thus elements requiring more relative attentions were identified.

SELECTION OF TESTS AND INSTRUMENTAL INVESTIGATION

To assess the condition of concrete in terms of its existing quality and acceptability of its strength, the tests are usually chosen considering the economy, damage to structure, speed and reliability. Core tests provide for a most reliable in-situ strength assessment, but causes damage to structure [4]. The non-destructive tests (NDT) such as Rebound Hammer test or Ultrasonic Pulse Velocity test provide indirect measure of strength and the quality of concrete through specified indices namely, rebound number index and ultrasonic pulse velocity (USPV) respectively. Estimation of strength of concrete in structure through these tests is possible when established correlation between the measured indirect indices and the strength is available. The correlation is again concrete specific. Reliable correlation between the strength and the suitable index can be established, by testing cores drilled from the concrete structure or cubes prepared from the same concrete. The drilled cores or the cubes are suitably tested for both strength and non-destructive test index for the purpose of generating data that is used in establishing the statistical correlation. In absence of such correlation, the concrete can be appraised at best qualitatively as `excellent', `good', `medium' or `doubtful'. The meanings of the term `excellent', `good', `medium' and `doubtful' are based on ultrasonic pulse velocity measured at site and are as per the nomenclature of IS 13311(part-1): 1992 and other reference literature [1]. A pull out insert can be placed in the existing concrete by drilling hole for the pull out rod and expanding sleeve. The pull out force can be reliably related to the compressive strength of concrete. There are a number of versions of this test and CAPO (Cut and pull out) test [1] is one such version. Thus pull out force and strength estimated from the calibration supplied by the manufacturer is a reliable means for estimating the in-situ strength of two concrete. It may be noted that since drilled-hole depth is very small, the damage done to the concrete is much less compared to core test. The tensile strength of cover concrete can be ascertained from Pull-off test which is similar to CAPO test. However, since experience of CAPO test is limited in India because of its very recent adoption, very limited core tests are also performed in this investigation in conjunction with CAPO test at nearly same location so that reliability of the CAPO test results can be verified independently.

In-situ carbonation test and cover measurement test gives an indication of the depth up to which carbonation has occurred and the extent of cover generally provided to rebar at site. These two tests together give information regarding carbonation being the potential cause of corrosion. Presence of free chloride being the other potential cause of rebar corrosion other than carbonation, test for free chloride is necessary to get information regarding the presence of chloride being the potential cause of corrosion. Half-cell potential test is adopted to gauge the risk of corrosion at present and corrosion rate test is adopted to estimate the future repair requirements.

Considering the objectives stated in earlier section, Rebound Hammer and Ultrasonic Pulse velocity tests have been adopted for assessing the quality of concrete in the structure. The carbonation test in conjunction with cover measurement and test for free chloride is adopted to diagnose the possible reasons of corrosion distress. Core and CAPO tests are adopted for assessment of strength and lastly half-cell potential test and corrosion test are adopted for assessment of corrosion risk and prognosis of corrosion distress.

Concrete strength and quality vary widely within a member along the depth, therefore relative position within the member has been chosen for all the elements in a manner so as to obtain unbiased sample locations [1, 5]. It is assumed that same concrete was used throughout the building. Hence for assessing concrete quality, locations were chosen randomly covering both inside and outside of the building, as well as, all the floors of the buildings as far as possible.

The appropriate number of tests (sample) is determined based on desired accuracy and cost [1,5]. The number of tests required for a desired degree of accuracy can be calculated using Stein's formula [5]. According to Stein's formula the number of tests n required for a given type of test is given by $n = (ts/SE)^2$. Where s is the sample standard deviation, SE is the sampling error and t is Student's 't' value for a particular confidence level desired. In this formula SE is expressed in absolute value. Expressing SE as a fraction of true value the formula can be rewritten as $n = (t\ C.V/\ SE)^2$. Where C.V is the coefficient of variation and SE is the sampling error expressed as a fraction of true value. On this basis appropriate number of tests was decided.

Detailed methodologies for most of the tests are available in literature and only the relevant tests and all salient results would discussed here. USPV tests were conducted in direct mode and core tests were performed on70mm (≥3×m.s.a) core as m.s.a for concrete is 20 mm. The in-situ strengths were calculated by IS 519: 1959 procedure as well as by BS-EN formulae. The difference between the estimated strength being relatively small between two methods, estimated results by former method was considered. The interpretation of results is obtained as per IS 456:2000 and BS-EN 13791. The equipment used for CAPO and pull-off tests was Germann Instruments A/S make. The in-situ cube strength is estimated from the pull-out force using the correlation provided by the manufacturer which was verified earlier through lab calibration. The carbonation test was done by phenolphthalein indicator spray test on freshly broken/drilled dry concrete. Cover depth was measured at the location where carbonation test was carried out by exposing the rebar to obtain carbonation to cover depth. Water soluble chloride content was measured by titration with $AgNO3$ using automatic titrator. The corrosion rate was measured by LPR technique using RAPICOR. The corrosion rate is expressed in terms of penetration depth in μm/year (10-3 mm/year). In addition, the half-cell potentials of the reinforcement and the resistivity of the cover layer are simultaneously obtained as complementary information. The Ag/AgCl reference electrode (RE) is used in the instrument.

Results of Investigation:

Ultrasonic pulse velocity results demonstrate that quality of concrete in about 42.5 %, are medium, while about 24.2% are good and about 33.3% are doubtful. Histogram plots of both USPV and Rebound Hammer results, exhibit that the concrete belongs to a single source. Core and CAPO test results indicate that characteristic strength of concrete is less than 10.0 MPa as opposed to 25 MPa adopted in design. Exactly 50% of location exhibit carbonation to cover depth ratio more than1 with risk of corrosion at least moderate. According to literature [7] threshold value of free chloride above which corrosion takes place is 0.025 % by mass of concrete. Free Chloride contents of 53 % of the samples are above the threshold value for corrosion to occur according to above criteria. The joint probability of finding a location with cover to carbonation depth being more than 1 and chloride level above threshold is 0.53×0.5=.265 i.e., 26.5% and correspond to risk of corrosion being high. The probability of finding a location with chloride concentration higher than threshold but, with carbonation to cover depth being less than 1 is also 26.5% with risk of depassivation being moderate. Similarly the probability of finding a location with ratio of cover to carbonation depth being more than 1, but chloride content being lower than threshold, is 23.5% and corresponds to moderate corrosion risk. The risk of no-depassivation is 23.5%. These risks are likely to change by worsening with time. The chloride content at a few locations it is quite high, but for other locations the variation in chloride content is small. With no external source, the chloride is internal chloride present as contaminant from ingredients right from the beginning. Since the concrete quality is also varying so are the chloride contents. The risk of corrosion can be ascertained from half-cell potential as per ASTM C-876 criteria and the current rate can be estimated from corrosion current density. Only in 3 out of 27 elements surveyed exhibit high risk of corrosion amongst the ones which are relatively less distressed now. Corrosion rates on the other hand are greater than 10 µA/cm2, for 25 out of 27 elements.

Remedial measures and Conclusions

As observed from visual survey, the damages due to rebar corrosion had occurred in patches; hence patch repair had been suggested at locations where cracks and spalling is observed. Concrete is deficient in soundness and strength hence improvement of soundness of concrete by injection grouting and strengthening by jacketing was recommended. Risk of depassivation is high to moderate in more than 75% of elements, hence to extend the residual service life of elements it is essential that ingress of moisture must be resisted. Effective water proofing of all elements are recommended. ASTM criteria for half-cell potential is an indicator of risk of chloride induced corrosion, however measure corrosion rate are quite high even though potential do not indicate the same. Thus both carbonation and chloride content are responsible for corrosion in this case. It is also concluded that poor quality of original concrete and it contamination in terms of chloride through ingredients reduces the service life of elements. Through this section a systematic methodology for investigation of corrosion distressed structure is presented.

INVESTIGATION OF PRECAST SEGMENTS

A number of pre-cast elements for tunnel lining produced and stacked at the casting yard exhibited cracks and their worthiness for use was under question. These elements 6 in number together would form a cylindrical segment of lining and the curing methods adopted included steam curing, membrane curing. Due to non-uniform heating of curved elements during steam curing resulted in crack in some other cases wax based curing compound pilled off early resulting in cracks and crazing. Objective of the investigation was thus formulated as below.

Objectives

Basic Objectives of the test scheme are: (i) to ascertain the uniformity, quality and strength of concrete. (ii)To assess whether the cracks are live and progressing or have stabilized and to ascertain that the cracks do not widens further due to multiple handling at site. (iii) To ascertain the general degree of depth of cracks. (iv)To ascertain the durability and reliability of cracked concrete segments vis-à-vis accepted segments.

Tests and Instrumental Investigation

The tests required to achieve the objectives stated earlier are Rebound hammer and Ultra Sonic Pulse Velocity Test (USPV) in conjunction with limited number of core tests for objective (i). Tell-tale strips test was used for objective (ii). Crack depth was estimated through Ultrasonic Pulse Velocity test by surface probing. In-situ permeability test such as ISAT for accomplishing objective (iv) was suggested. However, a visual observation preceded the instrumental investigation

Results of Tests Conducted

The colour coding used for recording the cracks in the segments during visual observations are green, blue, red double black and double red respectively for $w < 0.08$ mm; 0.08 mm $\leq w < 0.2$ mm; 0.2 mm $\leq w < 0.4$ mm; 0.4 mm $\leq w < 0.8$ mm and $w \geq 0.8$ mm respectively. Crack widths of most of the observed cracks ranged between 0.02mm to 0.8mm (practically none above 0.8 mm). Crack Depth of the all cracks was more than the cover depth (practically none less than cover). From the analysis of Ultrasonic Pulse Velocity (USPV) and Rebound Hammer test results it was concluded that concrete is of excellent quality in most the locations in the segments and good quality at other locations as evident from the USPV results. The USPV values ranges from 4.36 km/sec to 4.82 km/sec. Out of 180 test locations concrete exhibited a good quality only in 18 locations, at all other locations the concrete quality is excellent. It was further concluded that the concrete quality is uniform and homogeneous and can be treated as obtained from a same source.

Repair Strategy

According to the recommendation of ACI 224R-90, interpretation, acceptance and repair strategy for the drying and plastic shrinkage cracks observed in the segments have been formulated as follows. Crazing are hairline cracks that is less than 0.08 mm (003 in) wide, generally would have appeared on surface and are shallow. These cracks are cosmetic in nature. Cosmetic cracks can be left un-repaired, as repaired look of the surface may be aesthetically poor compared to un-repaired ones. A crack where aggregate interlock does not exists, such that transfer of stress across the crack is not possible, is a candidate for repair. In practice crack wider than 0.8mm (0.035 in) may be such a crack and is a candidate for repair. A second issue is related to durability and the crack width shall be sufficiently small to protect the rebar from corrosion. Based on these aspects the suggested limit of crack width for concrete exposed to humidity, moist air, soil etc., is 0.3mm (0.012 in) and those exposed to deicing salts, seawater and for the concrete in water retaining structure this limit is tighter. Cracks observed are non-structural cracks and will not impair the load transfer. Again the exposure condition being no worse than moderate, cracks smaller than 0.2mm in width can be left unattended. The width of 0.2mm is adopted, as is the allowable crack width as per contractual requirements. Thus for ensuring complete protection from durability point of view it is proposed to attend to the cracks wider than 0.2mm Routing and Sealing repair methodology proposed in ACI 2241.R-84 was recommended . It may be mentioned here that no cracks wider than 0.8mm has been encountered so far. Recommended sealant was expected to have low viscosity, anti-shrinkage property with appropriate setting time and resilience after hardening.

REMEDIAL MEASURES FOR COLUMNS WITH FAULTY COVER

Raker columns meant for supporting a natural draught cooling tower shell had their reinforcement cage misplaced during construction. The main vertical rebars encased in the helical spirals forming the cage have non-uniform covers because of the relative shift of the cage from the external surface of the concrete. The overall formworks were all aligned appropriately with no relative displacement what so ever, beyond specified tolerance limits, vis-a-vis required alignment and geometry. The cylindrical rebar cages were not concentric with the circular cross-section of the concrete columns due to the relative shift of rebar cage with reference to concrete column resulting in defect. The diameter of the cage is reduced in some cases, with the cover in one side being 270 mm and that in the diametrically opposite side being 45mm. Thus diameter of the rebar cage became 785 mm instead of 1020 mm, a reduction of in-core concrete diameter by 235 mm. The measured effective concrete section with displaced cage is reported to be 865mm. With this section the columns capacity against axial compression and moments has reduced. Similarly in case of some of other columns the cage has shifted outward and the capacity of the column against axial tension and moment have reduced. Thus to restore the capacity following strategies were proposed. Addition of vertical rebars at locations where effective core diameter has reduced and excess cover is existing. The new rebar shall be added with a clear cover of 40mm so as to ensure both, control of crack width and adequate capacity. The anchoring length inside the shell ring beam shall be as per original design. Application micro-concrete overlay with a bonding coat on the surface of the existing concrete to make up for cover where ever it is less to restore durability related performance as per original requirement was proposed. To restore the area of confined concrete envisaged in the original design and also to confine the newly added rebar appropriate fibre- wrap treatment can be applied at the surface of the column where the reduced core concrete diameter is reported. Such wrap shall extend up to the bottom of the ring beam. The columns were short columns with slenderness ratio less than 10. The inner and outer covers are denoted by ci and co respectively. The two worst cases were identified, namely' Case A: ci and co are 45 and 270 mm respectively; and, Case B:ci and co are 250 and 45 mm respectively. This resulted in reduction of d_s diameter of the circle having the steel bars on its perimeter thereby resulting in a reduction in the carrying capacity of the column. The lower cover can be effectively restored wherever required and the confinement can also be restored effectively by fibre-wrap as recommended later. Due to lack of symmetry of rebar arrangement the columns are weak in one direction thus capacity against uniaxial bending is considered for restoring the lost load carrying capacity of the column. This approximate methodology would not affect the performance against bi-axial moments. For high axial load as in this case, the ultimate load capacity $P_u > P_b$ and compression failure is expected. The design P_u in such case is given by following equation. [8].

$$P_u = \varphi \left[\frac{A_{st} \times 0.85 f_y}{\frac{3e}{d_s} + 1} + \frac{A_g \times 0.8 f_{ck}}{\frac{9.6he}{(0.8h + 0.67d_s)^2} + 1.18} \right] \tag{1}$$

Where φ is capacity reduction factor and is taken to be 0.75 for columns confined with helical reinforcements. e is the eccentricity, h is the diameter of the column cross-section, A_{st} is the main reinforcement and f_y and f_{ck} are characteristics steel and concrete strengths respectively. Thus d_s influences the P_u more significantly than A_{st}. For the case B where capacity is lower against axial tension and corresponding moments, the capacity can be assumed to be approximately proportional to A_{st} for a circular section with rebars symmetrically placed; Put is as given by following equation [16]:

$$P_{ut} = \frac{A_{st} \times 0.85 f_y}{1 - \frac{e - 0.5h + c}{jh}} \tag{2}$$

Where jh is the lever arm and c is the effective cover depth along tension side. Thus capacity is assumed to be proportional to A_{st}.

Fibre- wrap treatment is to be applied at the surface of the column where additional rebar are proposed to be inserted. This will ensure confinement and also ensure that, inserted bars which are not tied by rings, remains in position and are not dislodged from their position when stressed. Thus the treatment is recommended for even those columns where axial tension is also a concern in the context capacity restoration. The fiber-wrap shall be able restore the original confining pressure and accordingly the fiber-wrap is proposed. The confining pressure due to spiral helical rebar f_i is given by following equation[9].

$$f_t = \frac{2 \times 0.85 f_y A_{sp}}{d_s s} \tag{3}$$

Where A_{sp} and s are the area of spiral bar and s is the spacing of the bar respectively. All other notations on the right hand side of the equation are as defined earlier. For FRP thickness of t and acceptable design strength in tension of FRP being f_f , the confinement pressure is given by [10,11]

$$f_i = \frac{2 \times f_f t}{h} \quad (4)$$

Therefore thickness of FRP laminate t, consistent with f_i, and, d was selected for the design strength ff in tension of FRP.

CONCLUSION

Three different cases of investigations and remedial suggestions are presented in this paper and are likely to be useful in practice

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Bishwajit Bhattacharjee

Dr. Bishwajit Bhattacharjee is professor at the Department of Civil Engineering, Indian Institute of Technology Delhi, India. After obtaining B. Tech (Hons.) degree from I.I.T. Kharagpur in 1978, he worked for M/s Gammon India Limited for a short period of two years. Subsequently he obtained M. Tech. and Ph.D. degree from I.I.T. Delhi in the year 1982 and 1990 respectively.

His areas of active research interest includes Corrosion of rebar in concrete, High-performance concrete, Microstructure modeling of concrete, Chloride ingress, Water ingress, Service life prediction and Life cycle costing of concrete structures besides Condition evaluation and Health monitoring of structures. His other research interests are Energy Efficient Building Design, Sustainability etc.

CONCRETE JACKETING WITH SUPPLEMENTAL DAMPING – SIMPLE TO EXECUTE SEISMIC RETROFIT

Sandeep Shah

Abstract: This paper covers the seismic upgrade of a seven story non-ductile concrete framed building of early nineties vintage. Analysis results revealed that the structures did not have sufficient structural capacity to resist even a moderate earthquake. To ensure a higher level of safety, reduce the risk of exorbitant repair costs and minimize building downtime after an earthquake, it was intended that the seismic upgrade of the structural system will target the performance standard of 'immediate occupancy'. A dual stage approach was used to address this retrofit. The first part consisted of providing robust concrete moment frames in each direction using the time tested jacketing methodology. This ensured adequate strength and stiffness to the structure. The second stage involved adding fluid viscous dampers. The dampers provided supplementary damping thereby reducing displacements, story drifts and also the seismic demand on the moment frames. Additional supplementary damping also protected the building frame against excessive nonlinear deformations. The design scheme used a total of 44 dampers of various force capacities. Analysis results of the retrofitted block showed that the dampers dissipated significant portion of the seismic energy, reduced displacements and story drifts and limited the seismic nonlinear demand on the concrete members. This seismic upgrade methodology proved to be technically sound, easy to execute, less disruptive to the occupants and resulted in significant savings both in terms of time and cost.

Keywords: Seismic Retrofit, Fluid Viscous Dampers, Concrete Moment Frames.

INTRODUCTION

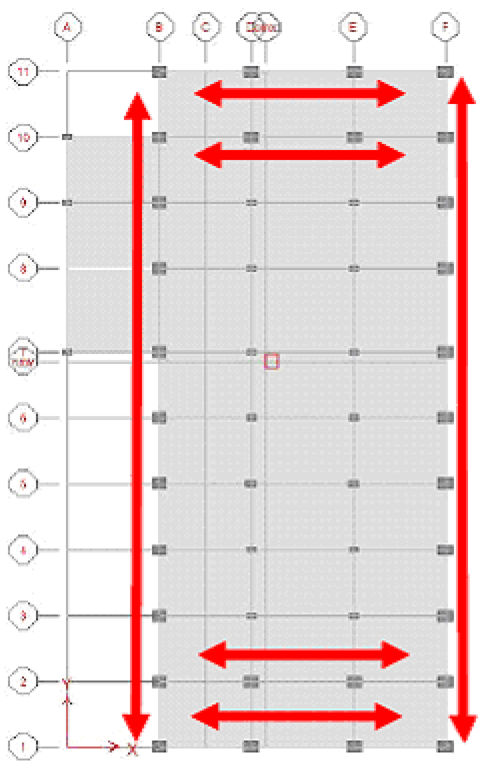
The concrete building intended to be seismically upgraded was built in early 1990's and analyses showed that the existing concrete members did not have adequate seismic capacity; many columns would fail even under a moderate seismic event. Additionally, the reinforcement detailing and distribution in this building was questionable given the fact that it was built in the early nineties and to older version of the seismic code. This building posed a higher risk to its occupants as its failure mechanism would be non-ductile. A two-stage retrofit was proposed to seismically upgrade this structure.

Stage-I: Increase the capacity of the existing concrete members by placing a reinforcement cage around the beams, columns, and joints and either pour concrete or use shotcrete. This methodology known as concrete jacketing has been time tested and used successfully for many lightly reinforced concrete buildings worldwide. The additional reinforcement bars were doweled to the existing members to engage the entire concrete cross section in resisting seismic loading. Due to time and cost constraints it was not feasible to jacket all the beams and columns, so only few frames were selectively chosen for concrete jacketing. Moreover it was planned that the second stage of retrofit would more than compensate for the deficiencies in balance frames. It was decided to strengthen only the perimeter bays in each direction. The perimeter locations provide the optimal locations for retrofit since they maximized the building rigidity and minimized its torsional response.

Stage-II: Addition of fluid viscous dampers (FVDs) to reduce seismic demand and story drifts. The dampers were strategically placed along the newly retrofitted concrete moment frames. These damper devices are very easily installed each taking only a few hours. The main advantage of fluid viscous dampers is that they do not add any additional force to the existing structural members since they are out-of-phase with the elastic forces. FVDs were successfully incorporated as the main component of the seismic retrofit strategy of this non-ductile building. FVDs also greatly helped in bringing down the retrofit costs and saving time as with this strategy it was not necessary to rip all the beams and columns for concrete jacketing.

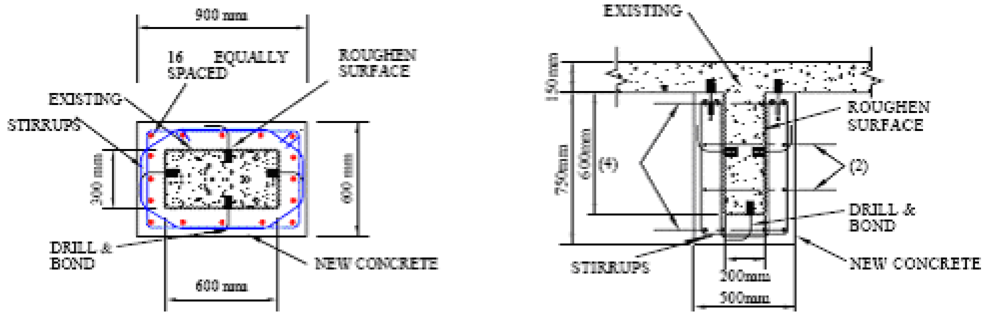
2. RETROFIT OF CONCRETE MEMBERS

Figure-1 shows the moment frames that were jacketed and Figure-2 gives out the structural detail for the concrete retrofit. For beams and columns, a reinforcement cage consisting of longitudinal reinforcement and ties was placed around members. The surface of the existing members was roughened to ensure bonding of the new to the existing concrete. Nominal dowels were drilled and bonded to provide redundancy in ensuring composite action between the existing and added concrete elements. The concrete used had a minimum compressive strength of 35 MPa. The longitudinal reinforcement provided the flexural capacity whereas the transverse reinforcement provided for shear capacity and confinement.



Typical floor and roof plan (______ indicates moment frames)

Figure - 1



a. Column b. Beam

Figure - 2

3 EVALUATION PROCEDURE TO ASSESS EFFICACY OF STAGE-I RETROFIT

An analysis was conducted to assess the response and performance of the stage-I retrofit. In this analyses model the retrofitted (jacketed) beams and columns were modeled as moment resisting and the remaining (existing) beams and columns were modeled as pinned. Two separate cases were considered.

3.1. Code-based design.

The strength (size and reinforcement) and drift requirements of the retrofitted concrete members was checked. The seismic loading was based on the static force procedure.

3.2. Performance-based design.

FEMA-356 approach was used to compute inelastic story drifts. It was intended to limit the drifts to approximately 1% (drift levels prescribed for immediate occupancy category). At these levels of drift, the members would experience very limited nonlinearity when subjected to seismic loading. Cracked properties were used for beams and columns. In analysis, mass contribution from concrete members, 150 mm slab, 150 mm perimeter precast walls and 150 mm stair and elevator walls was included. The non-structural mass was estimated to equal 1400 N/m2. Live load was uniformly distributed and had an intensity of 1900 N/m2. Figure 3 shows the analysis model of the building.

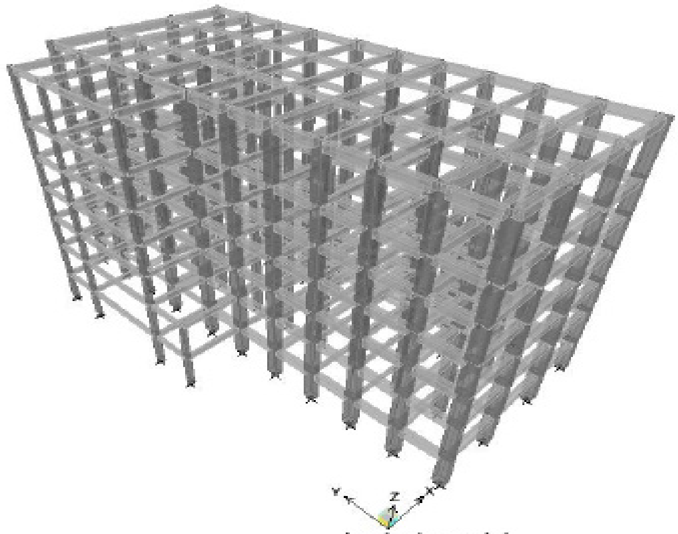


Figure - 3

4 RESULTS OF STAGE-I RETROFIT

4.1 Building mass and stiffness

Beam, column, and slab mass were directly computed by the program. The other components were added along the perimeter, on the slab, and at the concrete wall bay. Table 1 presents the computed seismic mass, and centers of mass and rigidity. Note that the building's center of mass and rigidity are closely located. All concrete diaphragms were modeled as rigid. At each floor the eccentricity between the center of mass and rigidity was approximately 1 m, corresponding to 4% offset in the x- and 2% offset in the y- directions.

Table-1

Building Mass			
Story	Mass in MN	Centre of Mass to Centre of Rigidity Offset	
		X-direction in M	Y-direction in M
L8	796	0.6	0.7
L7	866	0.7	0.6
L6	837	0.7	0.7
L5	837	0.8	0.7
L4	846	0.8	0.7
L3	892	1.0	0.7
L2	889	1.0	0.8

4.2 Dynamic properties

The dynamic properties of the building are shown in Table 2. The modal properties were based on the cracked sections of 50% of gross for beams and 70% of gross for columns. The fundamental mode in each direction alone captures over 85% of the building seismic mass. The analysis was conducted using 12 modes. Table 2 shows that the code requirement of capturing over 90% of building mass was met by the first 6 modes itself.

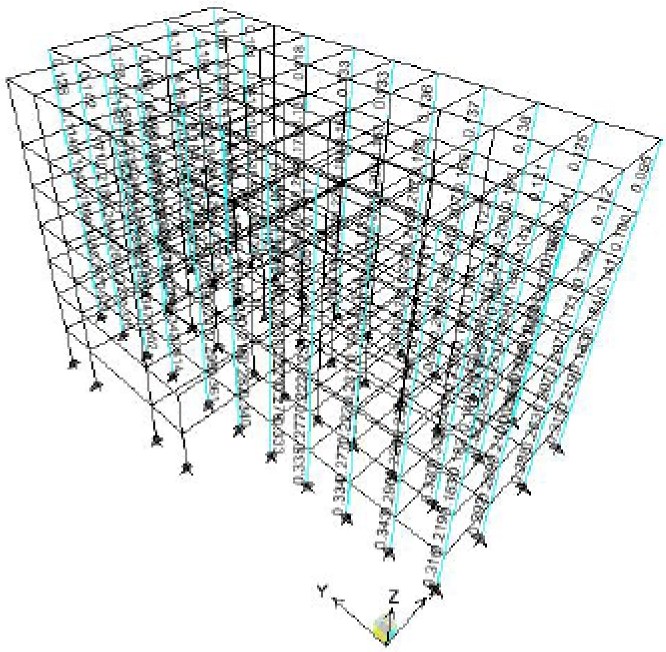
The modal responses in lateral and torsional directions are uncoupled because the concrete SMRF are nearly symmetric with respect to the center of rigidity.

Table-2

Modal Properties of the Building				
Mode	T in sec	Mass Participation		
1	1.34	88	0	0
2	1.18	0	89	2
3	1.02	0	2	87
4	0.41	7	0	0
5	0.38	0	6	0
6	0.32	0	0	6
Sum		95	96	95

4.3 Design check

The code load combinations were used to perform a design check of the building including static seismic loads. Figure 4 presents the results. Note that all members have a flexural demand to capacity ratio of less than unity. The design check was conducted for the moment frames members only.



DCR for concrete members

Figure - 4

4.4 Drift calculations

The largest story drift ratios (defined as the ratio of story drift divided by the story height) occurs at the first floor. The inelastic story drift ratios were computed and listed in Table 3. The maximum story drift ratios are over the immediate occupancy limit of 1%. Therefore it was concluded that by stage-I retrofit alone we do not achieve our target performance objective of immediate occupancy.

Table-3

Computed Inelastic Drift Ratios				
Direction	Δs, in mm	Δm in mm	H in mm	Drift Ration in %
X	8.5	51	3330	1.50
Y	7	42	3330	1.30

Response spectrum analysis was carried out to verify the computed inelastic drifts. Since this building is essentially a Single Degree of Freedom (SDOF) system in each direction, and since it will be designed to limit the inelasticity in the concrete members, this approach would apply.

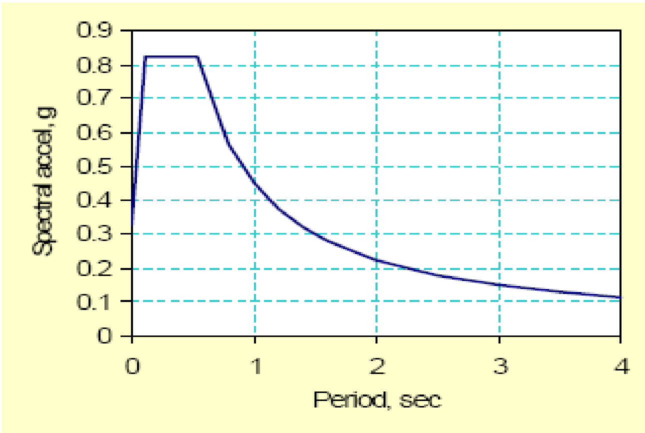


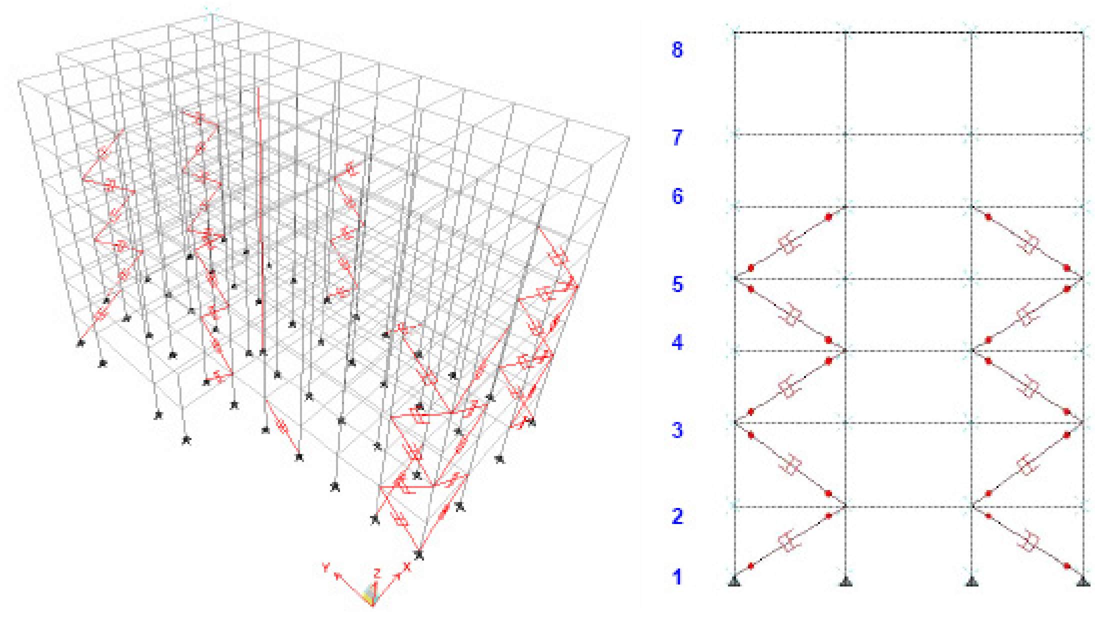
Figure - 5

The computed roof displacement in the x-direction for the building was approximately 240 mm, measured at the building corner with the largest drifts. The resulting maximum story drift ratios for the bottom story were approximately 1.8 % in the x- and y-directions; slightly larger than results from static analyses. This does not meet the intended performance expected from the retrofitted structure. In stage-II dampers were added to the building frame so as to reduce story drift ratios to below 1.0 % and also to limit seismic demand on the building. When no dampers are present all of the seismic energy is dissipated causing some or the other structural damage.

5 STAGE-II RETROFIT - DESIGN OF SUPPLEMENTAL DAMPING SYSTEM

5.1 Modeling

The analytical model for this analysis was similar to the last model with one exception, all members were assigned mass-less properties and the floor lateral and rotational masses as given in Table 1 were lumped at a point having 5% eccentricity with respect to center of mass. Since the center of mass at each floor is offset in the -x and +y directions from the center of rigidity, the additional 5% offset was placed in the same quadrant to maximize the torsional demand. Additionally, dampers were added to the model as shown in figure 6.



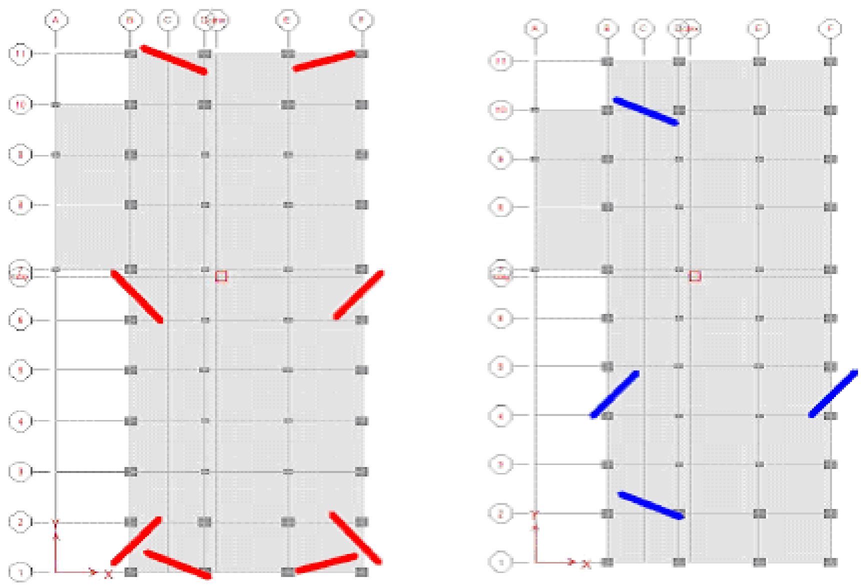
a. Schematic b. Typical elevation

Analytical model with FVDs added

Figure - 6

5.2 Damper locations and properties

Eight dampers (four in each direction) as shown in Figure 7, were used in second to fifth floors. Since the drifts were largest in first floor, 12 dampers were used at this level. The damping properties were selected to provide approximately 20% of critical supplementary damping at each level. Larger damping ratio was used at the first level to reduce the story drifts further.



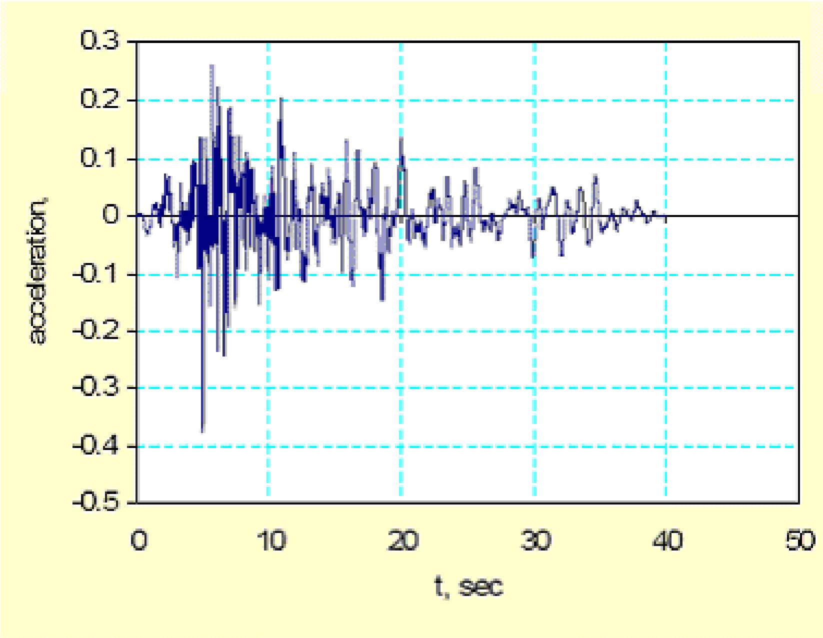
a. Dampers between ground and 6th floor (40) b. Additional dampers for the bottom floor (4)

Arrangement of dampers along the building frames

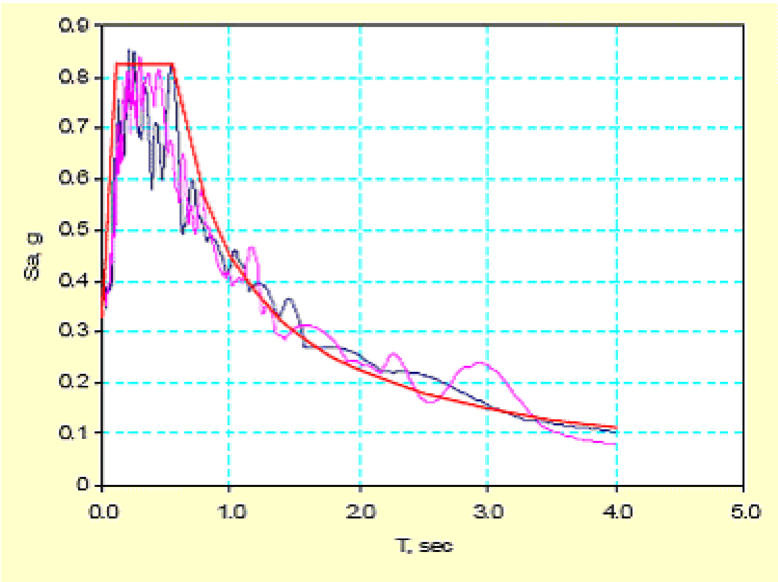
Figure - 7

5.3 Acceleration record

A pair of acceleration histories whose response spectra closely matched that of Figure 5 was used to perform nonlinear response history analyses. The resulting acceleration record and spectrum are shown in Figure-8. In analysis, the two components of the record were applied in x- and y-directions concurrently for two separate analyses.



a. Spectrum-compatible motion



b. Target and analyses spectra
Acceleration record used in analysis

Figure - 8

5.4 Performance based design methodology

In the performance based design approach, the retrofit is geared to obtain a desired level of performance during seismic events. For the building under consideration, FEMA 356 defines various performance levels for the concrete beams. The performance levels are based on the plastic rotation in these members. For the building under consideration, since nonlinear response is anticipated to be primarily concentrated in the concrete beams, plastic rotation values will correspond to similar values in the story drift ratios. Immediate occupancy is defined below 1% and Life Safety is considered between 1 to 2%. Immediate occupancy performance target was selected. Adding supplementary damping gives a solution much superior then only to having retrofitted RC frames. The retrofitted frames provide stiffness and strength whereas the FVDs add supplementary damping which helps in further reducing the story drifts and also the seismic demand on the building.

5.5 Response evaluation

The maximum computed displacements at the roof were 142 mm and 104 mm, in the x- and y directions, respectively as measured at the center of mass; see Figures 9 and 10.

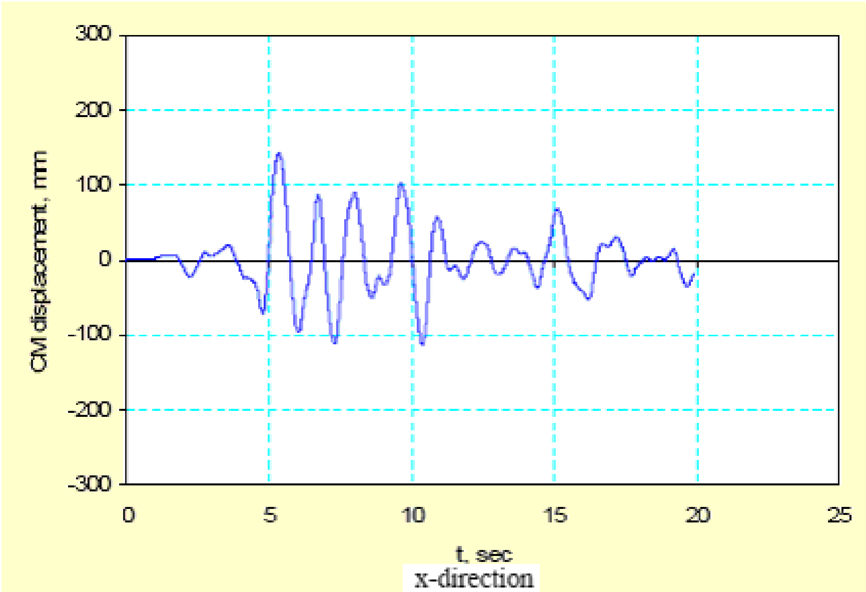


Figure - 9

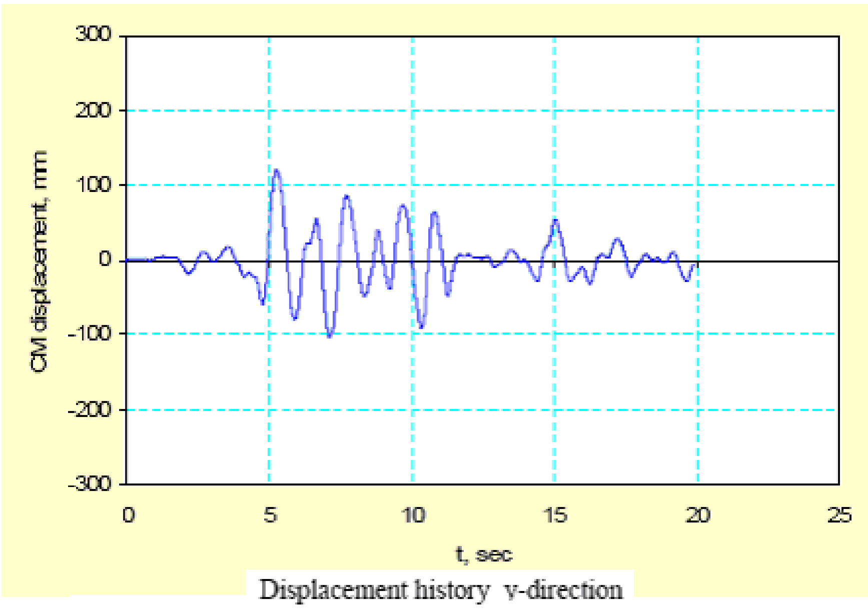
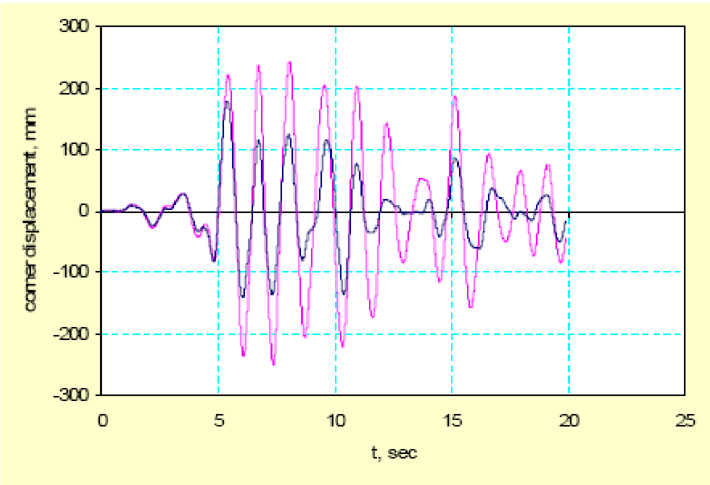


Figure - 10

The resulting story drifts are summarized in Table 4. The drifts are measured at the building corner where story displacements are the largest. As shown in Figure 11, addition of dampers has reduced both story displacements (and drifts).

Table-4

Computed Story Drift ratios in % from Non-Linear Analysis				
	Without Dampers		With Dampers	
	X	Y	X	Y
Roof	0.36	0.21	0.30	0.16
L7	0.42	0.26	0.40	0.21
L6	0.54	0.33	0.52	0.28
L5	0.78	0.42	0.66	0.36
L4	1.01	0.53	0.79	0.44
L3	1.39	0.78	1.00	0.63
L2	1.71	1.00	1.33	0.88

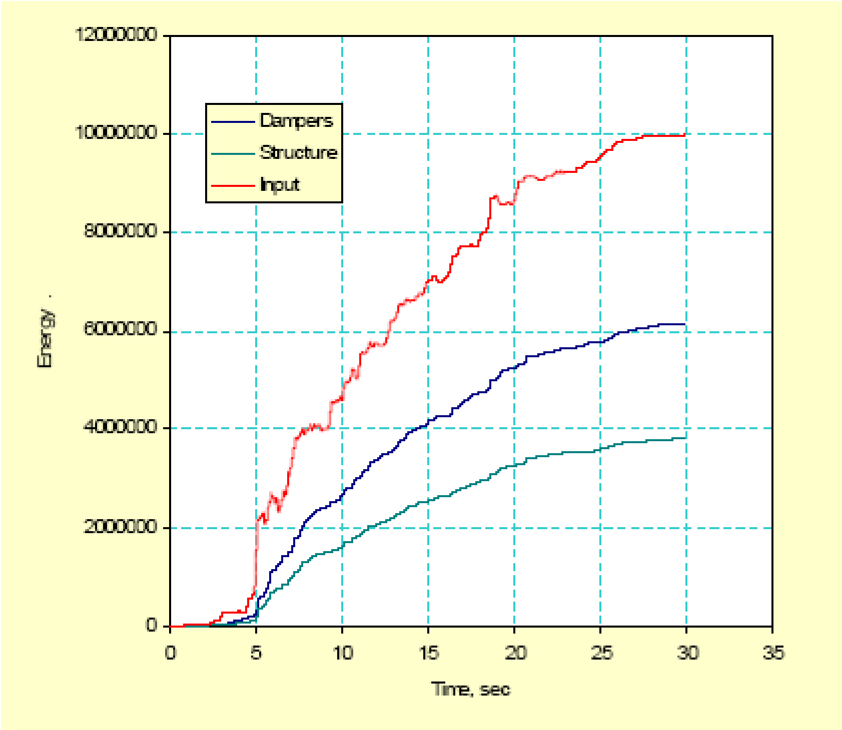


Center of mass x- displacement with(______) and without dampers (______)

Figure – 11

5.6 Energy Components

Figure 12 presents the normalized components of the seismic energy. Note that the dampers, indicated by the solid blue line, dissipate the most significant part of the input seismic energy. This is critical because if dampers do not dissipate this then this energy would be absorbed by the structure and cause yielding of concrete members resulting in structural damage to beams and columns.



Components of seismic energy

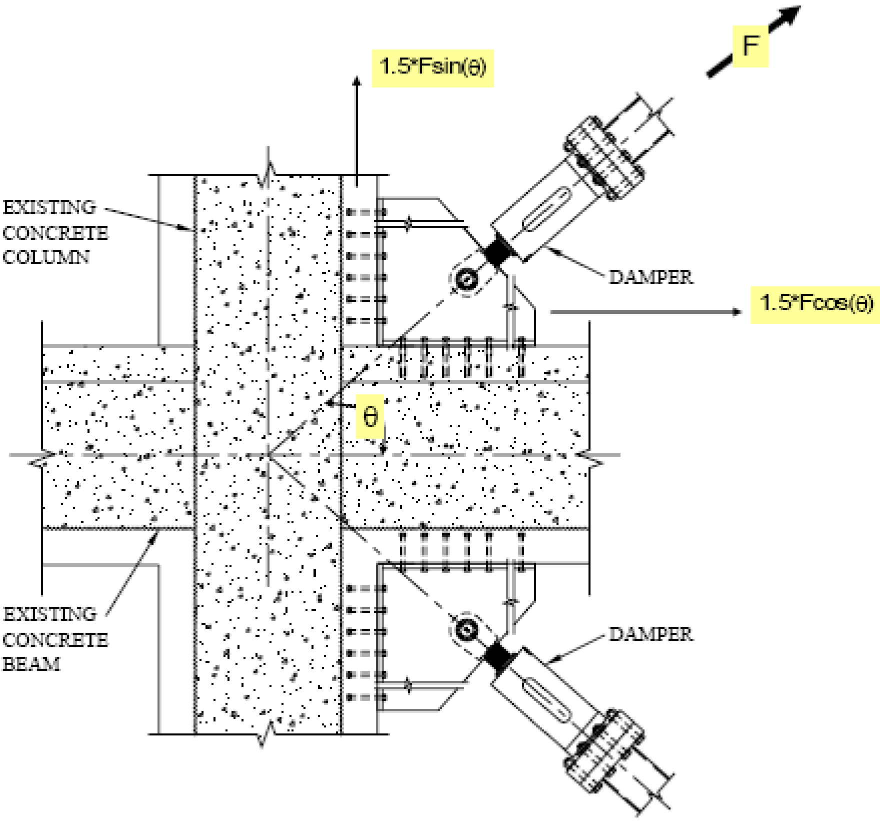
Figure - 12

5.7 Damper properties

Relevant damper properties are summarized in Table 5. Forty-four dampers are used for this retrofit.

Table-5

Damper Properties						
Damper ID	Floor	No.	C (KN-sec/mm)	α	Fmax (+/- KN)	Umax (+/- mm)
FVD 5	5-6	8	3.5	1	320	15
FVD 4	4-5	8	4.4	1	430	20
FVD 3	3-4	8	4.4	1	450	25
FVD 2	2-3	8	4.4	1	660	35
FVD 1	1-2	8	5.6	1	1010	40
FVD 1A	1-2	4	7.9	1	1340	40



An example of damper connection detail to existing and new concrete members

Figure - 13

6 CONCLUSIONS

This paper presents an easy to execute method of seismic retrofit of non ductile concrete frame buildings. Supplemental damping provides an economic, easy to execute and speedy solution for upgrading such structures.

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Sandeep Shah

Mr. Sandeep Shah is a professional engineer and Managing Director of Taylor Devices India Pvt. Ltd. and is based out of Gurgaon. He has undertaken Masters in Earthquake and Civil Engineering Dynamics from University of Sheffield U.K.

Sandeep's interests are in energy dissipation, seismic engineering, control strategies for dynamically sensitive structures, seismic upgrade and retrofit, high rise buildings and time-history analysis.

Other than design of many typical projects, Sandeep has undertaken many firsts in the country, Seismic upgrade of part of Apollo Hospital Delhi using fluid viscous dampers, design of the highest ATC tower in the country coming up at Delhi, this tower is also one of the most slender ATC towers in the world with an aspect ratio of 1:13, design of the first Tuned Mass Damper project in India, Design of damping system for the tallest building in the country i.e. Palais Royale, associated with Terminal-2 building at Mumbai airport where 70 fluid viscous dampers have been installed and many more.

ACI APPROACH TO REHABILITATION OF CONCRETE STRUCTURE: ACI CODE 562

Gajanan M. Sabnis

Abstract: The sustainable approach to the infrastructure rehabilitation is only by establishing ACI national standard for such work and was proposed by the author during ACI 364 initial activity. The approach was work hand-in-hand with the established ACI 318, Building Code and use modification so that engineers will understand it easier to create the practice. During the course, hands changed, more activities were undertaken and eventually ACI 562 was formed in early 2000 to perform this Task. The ACI 562 was recently released (March 2013) and is presented in this paper as the main theme.

This paper will explain the philosophy of this document and presents as to why a design code specific to concrete repair and rehabilitation is needed (to ensure safe structures). There are definite differences between the ACI 562 Repair Code and the ACI 318 and many guides to repair that are available and need to be explained if it is to be successfully accepted by the profession. It is also necessary to describe the governing philosophy and organization behind the creation of this document and to identify the scope and the development of various chapters of the new document.

The main motivation of this document was the Vision 2020 for Sustainable Concrete Structures by Strategic Development Council (SDC) of ACI. The document, Concrete Repair and Protection Vision 2020 Concrete Repair and Protection Vision 2020 was prepared by the SDC as an industry Critical Technology initiative. The main purpose was to provide a strategic plan for improvements in the Concrete Repair Industry, making the industry more efficient, effective, green and safe by the year 2020.

Vision 2020 was to create a repair/rehabilitation code to establish evaluation, design, materials and construction practices to raise level of repair/protection performance, establish clear responsibilities and provide Building Officials with means to issue permits for such work. There are challenges in the existing structures due to hidden damage and unknown structural conditions, lack of specific requirements for variations in repair practice, different levels of safety and reliability with no direction for building officials.

Various aspects of ACI 562, which is the most recent development in ACI will be presented to bring our Indian Community to higher level of standards and acceptance for new era of 3R's. In the last few years there has been a big awakening in India as well as has been demonstrated by the activity by ICI at various meetings, initiated by the author's keynote paper in ICI AGM in Allahabad in 2009.

Gajanan M. Sabnis

Dr. Gajanan M. Sabnis is Emeritus Professor of Civil Engineering at Howard University, Washington, DC, USA for more than 32 years and combined teaching, industrial and research experience totaling to over four decades, including heading his own Construction Management firm with offices in New York and Maryland for 20 years; his firms did a few large wastewater plants and transportation jobs. He obtained his B.E. from VJTI and M. Tech from IIT, Mumbai and later his Ph.D. from Cornell University, Ithaca, NY in 1967. After retirement in 2008, he splits his time in the USA and India to help improve the quality of concrete and infrastructure in India by using technology that he has developed or obtained from USA. He is a frequent speaker at the international conferences on Green Buildings and concrete. Dr. Sabnis built and lived in an energy-efficient award-winning home he built in concrete with several recycled construction materials. The house received national recognition. Dr. Sabnis has published a number of books and research papers read by engineers in many parts of the world. He is the recipient of numerous honors and awards including the James Berkeley Gold Medal from University of Bombay. He is the Distinguished Member of ASCE and Fellow of ACI, IEI, ICI, ACCE and the registered Professional Engineer in the US. He has been a nominated GC member of ICI since 2009. He was International Director for ASCE responsible for over 25 countries and was responsible for membership growth and active participant in the ASCE Board activities initiating new programs in that area. In ACI, he was actively involved in the International Activities and Chapter Activities Committee establishing many Institute Chapters in the US and abroad and was awarded the first Chapter Activities Award. Some of his notable publications are as follows:

- *"Green Building with Concrete: Sustainable Design and Construction",* Taylor and Francis, Boca Raton, FL (Indian Edition: 2012)
- *"Concrete Construction and Structural Evaluation: A Symposium honoring Dov Kaminetzky",* SP-285, (Co-editor), American Concrete Institute (ACI), Michigan, 2012
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STRENGTHENING OF IRON ORE FILTER UNIT'S HEAVY ENGINEERING SHED AT KUDREMUKH IRON ORE COMPANY LIMITED (KIOCL)

K. S Babu Narayan[1], Subhash C Yaragal[1], Katta venkatramana[1], Govind R Sangwai[2]

[1]Department of Civil Engineering, NITK, Surathkal, Mangalore
[2]Government Polytechnic, Osmanpura, Aurangabad

Abstract: KUDREMUKH IRON ORE COMPANY LIMITED, a Government of India enterprise, is Asia's largest iron ore mining and pelletisation complex and the country's major export oriented unit engaged in the business of exporting high quality iron oxide pellets and pig iron. Company's mining and beneficiation facilities were located at Malleswara, Kudremukh, and iron oxide pelletisation complex and Pig Iron unit are at the city of Mangalore, Karnataka. The 3.5 million-tonne capacity Pellet Plant complex comprises of the Filter Plant, wet grinding mills, one mechanized ship loading unit, 28-mw captive power plant, Roll Press, Pelletisation discs, Furnace etc.

The ore was being open-cast mined at Malleswara. Mixing the ore with water to give pump-able consistency slurry and then conveying it to Mangalore through pipeline was the practice. The slurry was being squeezed to separate water and ore, by vacuum filters to make it amenable for further processing.

Supreme Court suspended mining activities at Malleswara, Kudremukh recently. For M/s. KIOCL to sustain their operations ore had to be brought from elsewhere, when this practice began, the yield from Vacuum Filters drastically dropped due to change in consistency of the ore. To boost yields, change over from Vacuum Filter technology to Pressure Filter techniques were taken up by pilot plant studies, which confirmed the suitability of the change over.

Nevertheless, such a change over necessitated adequacy checks on the plant structural elements to cater to the enhanced loadings and conditions. This paper presents the structural adequacy checks and modifications to the plant building which were conceived, analyzed, designed, detailed and executed without hampering round the clock operations of the units.

Katta Venkatramana

Dr Katta Venkatramana obtained his BE (Civil Engg) degree from University of Mysore in 1981, ME(Ocean Civil Engg) from Kagoshima University, Japan in 1986 and Dr. Engg (Civil Engg) from Kyoto University, Japan in 1989. He worked as Postdoctoral Research Fellow at University of Oxford, UK during 1990-91 and later as Assistant Professor (1991-94) and Associate Professor (1995-2002) at Kagoshima University. Since 2002, Dr. Venkataramana is serving as a Professor at NITK, Surathkal. Currently he is the Head of the Department of Civil Engineering. His research interests include structural dynamics, earthquake engineering and offshore structural engineering.

FRP IN CIVIL STRUCTURES OF INDIA

Abhijit Mukherjee

Abstract: The Research on Civil Engineering applications of FRP in India has a history of nearly twenty years. Applications of the technology in infrastructure were initially a trickle. The first major project was after the earthquake of 2001 in Gujarat. In the last decade the technology has gained popularity and created a niche for itself. However, there are significant challenges to its progress.

My talk will open a discussion on experiences in research and practice; our achievements and challenges. Through the hour we shall try to identify the main challenges of the technology and possible ways of addressing them.

Abhijit Mukherjee

Professor Abhijit Mukherjee earned his PhD in Engineering from Indian Institute of Technology Kharagpur. He pursued post-doctoral work in Max Planck Institute of Materials Research, Stuttgart and Institute of Computer Applications in Germany. Professor Mukherjee has participated in the Educational Leader Program at the Institute of Educational Management, Harvard University in 2010. He has been in the faculty of Civil Engineering of Indian Institute of Technology Bombay for twenty years and Director of Thapar University for seven years. Presently he is the Institute Chair Professor and Dean Research and Development of Indian Institute of Technology Gandhinagar.

His research is on three pillars of Sustainable Structural Engineering, materials, monitoring and maintenance. He has more than 150 publications that are cited more than 1400 times with an h-factor of 25. He has supervised doctoral works of 18 PhD students and 5 are working with him now. He has been the principal investigator of collaborative research funded by internationally reputed organizations such as National Science Foundation, USA, Federal Ministry of Education and Research, Germany, European Union, and others.

Professor Mukherjee has received the ASCE outstanding research award. As recognition of his achievements in the field of Engineering, Indian National Academy of Engineering has conferred on Professor Mukherjee their Fellowship. He is also a Fellow of the Punjab Academy of Sciences for his contributions in Engineering Sciences.

APPROACH TO HOLISTIC REHABILITATION OF STRUCTURES

Aman Deep Garg

Abstract: The aim of structural rehabilitation design and execution is to achieve an acceptable probability that structures would perform satisfactorily during their intended service life. This paper aims at throwing light to basic approach for the assessment, strengthening and rehabilitation to resist gravity and earthquake forces with reference to Indian Standards including fundamental concepts of Structural Engineering. It discusses about the assessment of building defects, probable constructional errors, post construction finding of distress and commonly used techniques for the retrofitting along with reference to Indian Standards guideline for the same

Keywords: Design Strength, Lateral Force, Strain Compatibility, Strengthening, bracing, ductility, Durability, Grouting.

INTRODUCTION

The aim of Structural rehabilitation design and execution is to achieve an acceptable probability that structures would perform satisfactorily during their intended service life considering probable/ applicable forces (Gravity as well as lateral). With an appropriate degree of safety, they should sustain all the loads and deformations of normal construction and use and have adequate durability and adequate resistance to the effects of misuse and fire.

The rehabilitation of a facility is suggested when signs of distress are noticeable or deficiency is found in meeting the intended serviceability/ strength requirements for its use. The first step in this regard is the visual survey and collection of data. Suitable Non-destructive tests (NDTs) are planned for the assessment of the structure/ its material components. Structural analysis is performed to assess the structural capacities of its component as well as the required capacities to meet the design requirements. When viability of rehabilitation is established, the structural design and detailing is done using appropriate structural systems, materials and techniques. Effective supervision and specified execution quality must be ensured to get the expected benefits/ objectives of the rehabilitation efforts.

PROCESS FOR REHABILITATION OF STRUCTURES

The suggested process for the rehabilitation starts with the assessment of structural elements and requires visual survey and detailed assessment data. Compiling all the documents of original building/ structure should be the first and important activity. Following data/ documents are the minimum requirement for structural assessment:

a) Architectural Drawings and year of construction,

b) Structural Drawings including foundation details and design calculations,

c) Geo-technical report,

d) Construction specifications,

e) Any past re-modeling, repair or rehabilitation,

The structural system of the building should be assessed and classified into the prevalent system of Load bearing, Moment resisting Frame or dual system. Design for the rehabilitation includes design based on existing system for durability, constructability and service use as a whole.

The mechanical properties of structural materials and components are assessed based on the available construction documents and as-built conditions for the particular structure. Where such information fails to provide adequate information to quantify the material properties, it should be supplemented by appropriate material tests and NDTs. The nature and extent of distress/ damage/ cracks should be assessed and classified as ***Insignificant, Slight, Moderate, Severe and Very Severe.***

To accurately assess the damage present in the structural elements, it is necessary to distinguish between environment conditions induced cracks and stress induced cracks. For stress induced cracks, it is important to distinguish between flexure cracks or shear cracks. It is also necessary to identify the cracks that may indicate lap-splice or anchorage slipping. Cracks widths should be determined in order to assess the severity of damage of the structural and non-structural members.

A detail registry of all evidences should be drawn including relevant data such as nature of defect or damage and their location, extent, length, width, thicknesses, as applicable.

Based on the assessment of material properties and damages, the next step is to work-out the status of building/ structure under proposed usage and probable loads. Based on specific load standards as for in India, Indian Standards ***IS875*** and ***IS1893*** may be used to assess the design loads, which may be classified as below:

Types of Loads, Forces and Effects

- Permanent Load : Self Weight, Dead Loads etc,
- Variable Load : Live Loads, Wind, Traffic etc,
- Temporary/ Accidental Loads : Earth Quake, Impact, Erection, Fire, Explosion, etc,
- Miscellaneous Other Loads : Temperature, Shrinkage, Fatigue, etc

Recently published Indian Standard **IS 15988:2013** (Seismic Evaluation and Strengthening of existing reinforced concrete buildings-guideline) give modification factor for the lateral load considering the reduced usable life of structure. The usable life factor U is to be multiplied to the lateral force (base shear) for new building as specified in IS 1893 (Part 1). U (modification factor) is to be less than 0.7 and is determined as

$$\mathbf{U = (T_{rem}/ T_{des})^{0.5}}$$

Where

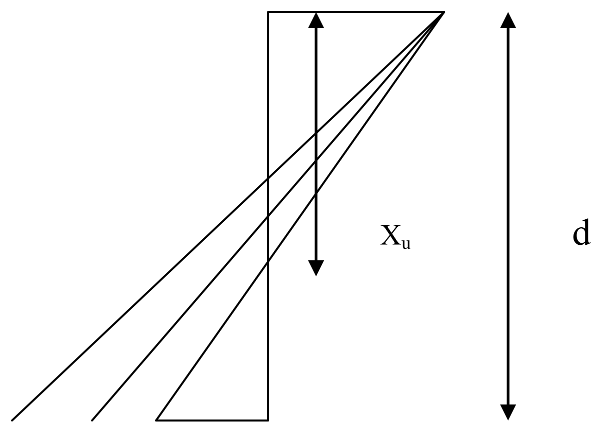
T_{rem} = Remaining useful life of the building

T_{des} = Design useful life of the building

While designing for rehabilitation considering appropriate applicable loads and force, it is very important to analyze the effective load path, forces and stresses on structural members and the strengthening design of the structure for gap in the required strength and the available in the existing. Emphasis should be given to strain compatible design of structural strengthening so as have composite behavior of materials and stresses within limits.

Maximum percentage of reinforcing steel in RCC structural member (based on fundamental principle derived from strain analysis):

For safe design of RCC, Reinforcing steel should be yielding when ultimate concrete strain is reached.



Therefore

$X_u / d \quad = \varepsilon_c / \varepsilon_c + \varepsilon_s$

$= 0.035/ [0.035 + (fs/Es)]$

$= 0.035/ [0.035 + (0.87\ fy/200000)]$

Or in conservative side

$\mathbf{X_u / d = 500/(500 + fy) = 0.0259 \times P_t \times (fy/ f_{ck})}$

Or P_t (Max) = 0.04 f_{ck} **(for fy = 500)**

Or P_t (Max) = 0.05 f_{ck} **(for fy = 415)**

Minimum percentage of reinforcing steel in RCC structural member (based on fundamental principle derived from strain analysis):

The minimum percentage of reinforcing steel is governed by the requirement of Moment of Resistance. The tension reinforcing steel should be able to take full stress when Concrete crack in tension and transfers the stress.

i.e. $\mathbf{A_{st} . f_s . (Lever\ Arm) > f_{ct} . Z}$

or $P_t (Min) . (b.d/100) . (087\ f_y) . LA > f_{ct} . (b.d^2 / 6)$

(Consider LA as 0.85 d)

P_t (Min) > 22.5 (f_{ct} / f_y)

Say on conservative side **P_t (Min) = 25 (f_{ct} / f_y)** where f_{ct} is the concrete tensile strength.

METHODS OF STRENGTHENING

The structural rehabilitation design and detailing is required to strengthen the structural members/ elements. Some of the notable methods of strengthening that may be used are as below:

a) **Addition of Infill Walls**: The construction of infill walls within the frames of the load bearing structure aims to give significant increase in strength and stiffness. This method can be applied in order to correct design errors in the structure specifically, when a large asymmetric distribution of strength and stiffness has been recognized.

b) **Addition of New External Walls**: The strengthening, in some cases by adding concrete walls, can be performed externally. This can often be suggested for functional reasons like when the building must be kept in operation during the strengthening works. New cast-in-situ concrete walls can be planned and placed outside the building to resist partially or the total seismic forces induced in the building.

c) **Addition of Bracing Systems**: The construction of the bracings within the frames of the load bearing structure aims at increasing the stiffness and a considerable increase in the strength and ductility of the structure.

d) **Construction of Wing Walls:** The construction of RCC wing walls in the existing columns with proper connection helps in increasing the stiffness and strength of the structural members and may be planned to balance the asymmetric distribution of stiffness.

e) **Strengthening of Weak Elements**: The selective strengthening of the weak elements/ members of the structure aims to avoid a premature failure of critical elements of the building/ structure and to increase the ductility of the structure. This method is usually applied to members by way of concrete jacketing, steel encasing or using FRP jackets.

f) **Enhancement of Ductility**: When the stiffness and the general strength of the structural members is not critical, the ductility enhancement at the critical location of plastic hinge formation i.e. beam column junctions etc. is done to gain the advantages of ductility and thereby enhancing the mechanism of resisting the lateral forces.

GENERAL METHODS OF REPAIRS

During structural rehabilitation, repair of structural and non-structural members/elements is also required to be dealt with. Some of the methods of repair that may be used are as below:

a) For Superficial repairs: Cement plaster application over treated surfaces.

b) For General non-structural or minor structural repairs

- Cement based polymer modified mortar over cement slurry coating.
- Epoxy mortar treatment over epoxy primer coating with wire mesh reinforcement.

c) For Principal repairs on members with significant strength weakness

- Repair with fiber wrap technique at junctions of Beam-Columns

- Sealing of structural cracks by epoxy grout injection
- Sealing of honeycombed portions using non-shrink cement grout.

CONCLUSIONS

The structural rehabilitation need to be planned with thorough assessment and inputs data. The structural analysis is to be performed using appropriate method and applicable standards considering all applicable loads. The demand of the strength is compared with the existing strength and the deficiencies, if any, need to be handled by choosing appropriate system of strengthening, either by increasing capacity of the existing structural members or by adding new structural supports or by stiffening or by increasing the ductility as per the best feasible solution. The state of structural members/ elements (as carbonation, rusting, de-bonding of reinforcement, deflections etc) is also important factor and should be effectively dealt with. The Indian Standards such as IS 456, IS800, IS 1893, IS4326, IS13920, IS 15988, SP25 and other technical documents throw light to deal with different conditions and for performing the structural design calculations for effective and reliable rehabilitation design.

REFERENCES

1. IS15988:2013 - Seismic Evaluation and Strengthening of existing reinforced concrete buildings-guideline.
2. SP 25: 1984 Handbook on causes and prevention of cracks in buildings
3. IS 456:2000 – Plain and Reinforced concrete- Code of Practice

Aman Deep Garg

Mr. Aman Deep Garg Competent professional in the construction industry with about 15 years of experience of multifaceted domains especially Civil/ Structural design, Project Management, Engineering and Coordination. Main Strength lies in innovative approach towards engineering challenges and bringing cost effective design to meet the customer expectations.

A throughout meritorious student and an Engineering graduate from REC Kurukshetra (now NIT) and PG diploma in Construction Management from NICMAR, has served reputed organizations like NMSEZ Pvt. Ltd. (A Reliance group Company), DHV – GEC BV (A Dutch consultancy organization), Bureau of Indian Standards (National Standardization Body), HSCC (India) Ltd (A Government of India undertaking). With multifaceted experience in hand, founded 'Creative Design Consultants & Engineers P. Ltd' in 2009 and presently working on reputed national and international projects.

The appreciations throughout the career have been motivating to contribute the state-of-art contributions in the construction industry. Besides contributing in the revision of the important Indian Standards like National Building Code 2005 and IS 800:2007, technical papers have been contributed for Potential utilization of Fly-Ash, Construction Safety, Concrete Durability and many others in National and International conferences.

Presently represent as expert member on the following BIS committees:

- CED 2/P1: Panel for Work Relating to ISO/TC 71 & 74 of 'Cement and Concrete Sectional Committee'
- CED 2:2 : Concrete Subcommittee of 'Cement and Concrete Sectional Committee'
- CED 2:2/P5 : Panel for IS 456 and IS 1343 of 'Cement and Concrete Sectional Committee'
- CED 29: Construction Management (including Safety in Construction) Sectional Committee

He may be contacted at
aman@ccepl.co.in, aman.str@gmail.com

POST FIRE ASSESSMENT AND REHABILITATION OF STRUCTURE- A CASE STUDY

S. K. Singh

Abstract: The concrete buildings and other infrastructure has been seldom assessed for damages due to fires. Fortunately, even after a severe fire, concrete structures are often capable of being repaired rather than demolished. This provides substantial savings in cost and also savings in consequential losses which includes relocation cost of occupants. The post fire assessments are required before any specified repairs. This paper has made an attempt to describe the importance of structural engineers and material scientist in the post fire damage assessment of concrete structures.

Keywords: Assessment, Fire, Post fire investigations, Rehabilitation, NDT, Case Study

INTRODUCTION

In recent years a number of notable fires have occurred in buildings and commercial establishments due to electrical short-circuiting in India. These fires have caused huge loss of properties including several lives. This occurrence of fire is likely to rise as the number of tall buildings are coming up in mega cities without adequate fire detection and extinguishing system. In addition, no emphasis on systematic post fire investigations of damaged structures is given in India to ensure structural safety and enabling appropriate repairs to be planned and executed. Concrete buildings are most likely to be subjected to fire include private and public buildings such as offices, hotels, hospitals, warehouses etc.. Fortunately, even after a severe fire, concrete structures are generally capable of being repaired rather than demolished. It is because of its satisfactory performance against fire exposure. They are usually repaired and returned to service barring in a few cases, that they are partially demolished specially roof slabs and replaced with new one. In the event of a fire, the focus is on immediate measures for securing public safety. In India, the fire brigade department has take control of building in case of occurrence of fire and allows making it functional only after detailed investigations and assessment of the stability of the structure by reputed agency. The immediate interest of owner is to find the most appropriate and cost effective solution for repairing the structure.

CONCRETE STRUCTURES EXPOSED TO FIRE

In general, concrete is considered to be of non-combustible material which performs very well when exposed to fire. However, due to heterogeneous nature of concrete, each material present in it interacts differently at elevated temperature; hence the properties of concrete as whole may change radically when exposed to fire. The mechanical properties such as strength and modulus of elasticity are significantly reduced during these exposures. This may result into undesirable structural failures. Therefore, the properties of concrete exposed to fire are very important for determining the load carrying capacity of structural elements. Also there are changes in chemical and physical properties. The dehydration such as the release of chemically bound water from the calcium silicate hydrate (C-S-H) becomes significant above about 600°C. The dehydration of the hydrated calcium silicate and the thermal expansion of the aggregate increase internal stresses and micro-cracks are induced in the material above 300°C. Calcium hydroxide $Ca(OH)_2$, which is one of the most important compounds in cement paste, dissociates at around 530°C resulting in the shrinkage of concrete causing cracking and crumbling. Therefore, the effects of high temperatures are generally visible in the form of surface cracking and spalling. Some changes in colour may also occur during the exposure. The changes produced by high temperatures are more evident when the temperature exceeds 500°C and most significant changes have been found in concrete at this temperature level which has been considered irreversible.

The type and properties of aggregate also play an important role on the properties of concrete exposed to fire. The strength degradations of concretes with different aggregates are not same under elevated temperature because of different structure of aggregates. Quartz in siliceous aggregates polymorphically changes at 570°C with a volume expansion and consequent damage whereas limestone aggregate in concrete, $CaCO_3$ turns into CaO at 800–900°C and expands with temperature. Shrinkage is also observed due to the decomposition of $CaCO_3$ into CO_2 and CaO with volume changes causing destruction of concrete matrix (1-7).

The changes in the concrete matrix when exposed to fire calls for systematic study of characteristics of concrete exposed to fire as the residual properties of concrete help in determining the best rehabilitation strategy. The physico-chemical processes of concrete exposed to fire is summarized below along with Figure 1.

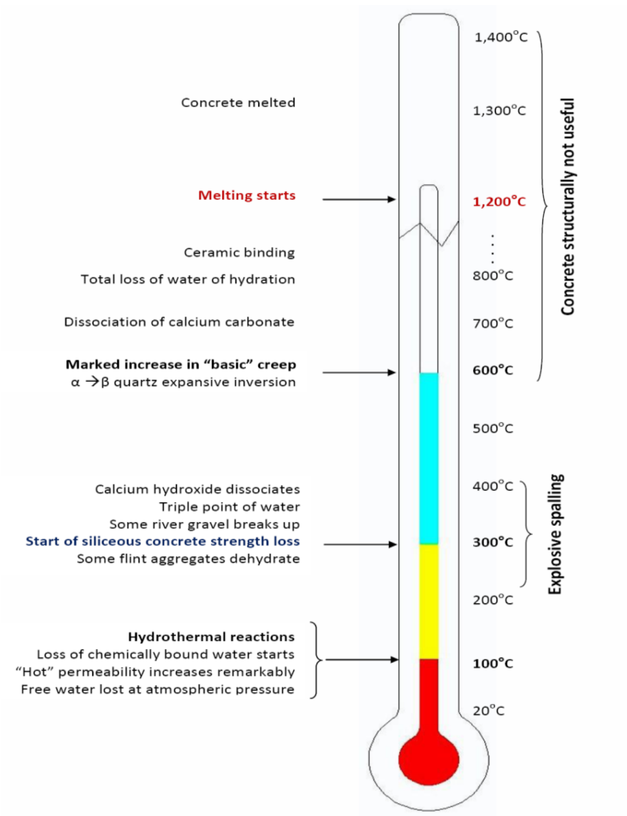


Figure 1 Physico-chemical processes of concrete exposed to fire

1. **30–110°C**: The evaporable water and a part of the bound water escapes. It is generally considered that the evaporable water is completely eliminated at 120 °C.
2. **110–180°C**: The decomposition of gypsum (with a double endo-thermal reaction, the decomposition of ettringite and the loss of water from part of the carbo-aluminate hydrates take place.
3. **180–350°C**: The loss of bound water from the decomposition of the C-S-H and carbo-aluminate hydrates take place.
4. **450–550°C**: Dehydroxylation of the portlandite (calcium hydroxyde).
5. **700–900°C**: Decarbonation of calcium carbonate. All the calcium hydroxide is converted to calcium oxide and the C-S-H converts to an anhydrous calcium aluminum silicate. These reactions cause a decrease in volume which leads to cracking in the paste.
6. **Above 900°C**: further decomposition of aggregates. Concrete it starts burning.
7. **700–900°C**: Decarbonation of calcium carbonate. All the calcium hydroxide is converted to calcium oxide and the C-S-H converts to an anhydrous calcium aluminum silicate. These reactions cause a decrease in volume which leads to cracking in the paste.
8. **Above 900°C**: further decomposition of aggregates. Concrete starts burning.

The significant loss in strength of reinforcing bar is observed at high temperature resulting into decrease in stiffness of structural members which is responsible for excessive residual deflections. However, recovery of yield strength after cooling is generally complete for temperatures up to 450°C for cold drawn bars and 600°C for hot rolled drawn bars. Above these temperatures, there will be a loss in yield strength after cooling.

The effect of high temperature is more critical on pre-stressing steel than on reinforcing steel.
At temperatures of 200 - 400 ºC, steel pre-stressing tendons show considerable loss of strength (>50% loss at about 400°C). In terms of reuse, a more important factor is the effect of heat upon the tension of the steel. Loss of tension may be contributed to by loss of elastic modulus in the concrete, increased relaxation due to creep and non recoverable extension of tendons.

POST FIRE ASSESSMENT OF DAMAGED CONCRETE STRUCTURES

The aim of post fire assessment of a fire damaged concrete structure is to suggest appropriate repair and rehabilitation methods. This also helps in estimation of extend of damages and decisions making for requirement of extend of removal of fire damaged concrete including whether demolition of elements or the whole structure is required.

Assessment of the extent of damages in structure can be achieved effectively through detailed visual/ physical examinations with help of light hammer at macro level and testing. The observations of the visual inspection in conjunction with testing provide the basis of damage assessment and help in deciding of an appropriate rehabilitation / strengthening techniques for damaged structures. To ascertain the degree and extend of damages in structure due to fire, detailed appraisal of structures along with analysis and material characterisation. The post fire damage assessment and rehabilitation of structures involves following stages:

1) Initial appraisal and estimation of fire severity

(a) Documentation of structures

(b) Visual inspection of distress mapping
(c) Fire spread estimation & maximum temperature attained due to fire
(d) Assessment of fire severity & damages
(e) Characterization of damages in terms of structural & non-structural

2) Testing and Detailed investigations
(a) Non-destructive evaluation of structural members element
(b) Core extraction & its evaluation
(c) Materials Characterization, collection of samples & its testing
(d) Assessment of reduction in strength & ductility of structural reinforcement
(e) Checking of structural design & drawing
(f) Analysis of data and its interpretation in terms of structural integrity & stability

3) Design of repairs to structural elements and protection measures
(a) Based on interpretation of the data collected during investigations, an appropriate repair design system is being evolved for rehabilitation of fire damaged portions
(b) Active/ passive fire protection measures for the building

4) Implementation of structural repairs
(a) Repair drawings and methodology
(b) Technical specifications, bill of quantities etc
(c) Quality control and quality audit during rehabilitation

5) Structural integrity and stability tests after rehabilitation

A FIRE DAMAGED BUILDING- A CASE STUDY

A post fire investigations of a fire damaged office building namely IOCL Bhawan at Noida and remedial measures including fire protection system is commissioned immediate after one day of fire. The building under investigations is a reinforced concrete (RC) moment resisting frame system having basement with 11 floors. The fire took place at the fourth floor of the building. It has caused charring of furniture and services, massive damages to structural elements, spalling of concrete etc. on roof slab of fourth floor including beams and columns.

The detailed visual inspection, mapping of distresses in terms of structural & non-structural, estimation of fire spread, assessment of fire severity and assessment of damage to structural elements has been carried out to assess structural integrity & stability of fire damaged area and to arrive at appropriate rehabilitation measures, detailed survey, non destructive testing, core extraction and its evaluation & characterization of materials including chemical analysis have been undertaken. Further, to know the health of structure in whole, ambient vibration measurement has been undertaken and analyzed.

Assessment of Fire Damages in Structures

The major fire causes extensive damage to concrete as well as to structural & non-structural members of the building. After a fire, it is imperative to assess the damage through survey of damaged area, examination of debris, debris characterization etc. The detailed study of fire-damaged area indicates that initiation of fire in building has occurred at one end near column at fourth floor. Further, the smoke and hot gases travelled in both directions of, raised the temperature of present combustible and hazardous material, mainly wooden furniture, cubical, wooden partition, files and filing cabinet, carpet etc. The rise in temperature of the combustible material due to convective heat currents and radiations helped in the growth and the development of fire. After the ‘flash over conditions’ near AHU room, the flame also travelled in all directions through the opening thereby accelerating the spread of the fire.

From the inspection of debris & assessment of fire damaged area, it was estimated that temperature during fire varied between 150^0C to 950^0C at different locations with fire severity of more than one and half hours in terms of standard time-temperature curve.

Visual Inspections and Distress Mapping

The whole structure is examined visually to assess the extent of damages. The predominant structural damages such as spalling of concrete beyond cover of slabs, cracking and delamination of slabs, deflection of slab panel, spalling of concrete cover from the beams and damages with lesser intensity to columns, deterioration and spalling of mortar in brick work of fourth floor, corbel and projections etc. have been observed. The visual inspection is summarized as below and shown in Figures 2-10.

- The severe damage to wooden doors, wooden wall lining, false ceiling, cables etc. has been found in corridor at fourth floor. The soot, traces of oil & grease have been found in all the rooms at fourth floor and of lesser intensity in stair-cases and fifth floor rooms.
- Crazing cracks are observed predominantly all over the roof slab concrete as well as on plaster. On tapping the plaster & concrete surface, hollow sound is observed which indicates separation of plaster and cover concrete from the slab.
- The excessive deflections were also noticed in a few slab panels.
- The ceiling plaster along with some part of concrete has peeled off at several locations over a wide area. Reinforcements were exposed in several locations considerably and partially at few locations.
- Disintegration of concrete has been found at several locations in beams, columns and slabs. The severe sectional loss is also found at a few locations.
- The yielding of reinforcement due elevated temperature has also been noticed at few locations.
- Cracks are visible on roof slabs vary from 0.10 mm to 0.50 mm. The widths of cracks are as high as 22 mm.
- Major cracks and delamination of roof slab are also observed.
- Internal delamination of slab is also observed which was further confirmed by core extraction from slab.
- While examining the fifth floor, it was seen that to provide proper slope, plain cement concrete(PCC) layer which is to the tune of 50 mm -130 mm thick, causing excessive overloading on the floor slab and subsequently on structural members. Further, floor tiles were found delaminated almost all the places in west wing.
- The thickness of roof slab has been reduced to 55 mm to 75 mm at several locations due to severe spalling particularly near AHU room of west wing.
- Surface Soot (black carbon) were deposited predominantly in building and on the stone cladding (Dhaulpur) on external facia of building causing discoloration which is usually washable by water jetting/ sand blasting.

- A few stone cladding on external facia is found delaminated and spalled from the surface due to fire exposure even though they are properly anchored with the surface. Further, it was observed that a few stone cladding has debonded with the surface causing falling hazard.

Figure 2. Severe fire damages at 4th floor of the building

Figure 3. Another view of fire damages at 4th floor of the building

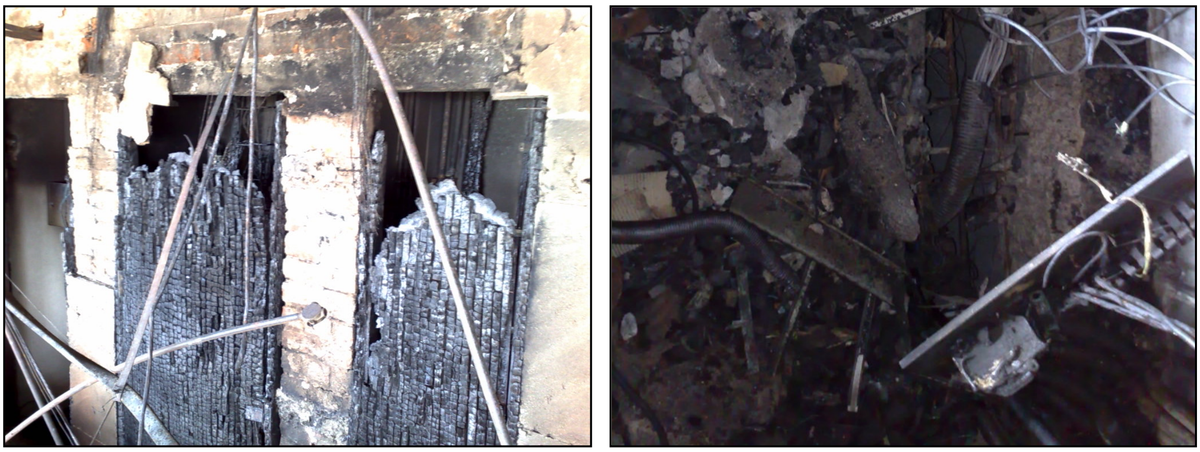


Figure 4. Severe damages to service shafts at 4th floor

Figure 5. **Delamination of tiles and cracks at 5th floor of building**

Figure 6. Damages in other part of building due to smoke & hot gases

Figure 7. Delimitation of stone cladding and soot deposition on fascia of building

Figure 8. Presence of Highly Combustible Materials at 4th Floor

Figure 9. Spalling of Cover Concrete in Roof Slab of 4th Floor

Figure 10. Severe Damages/ Delamination of Slab at 4th Floor

Non-Destructive Testing

Detailed appraisal/investigations of fire damaged structure is utmost important which includes examination of concrete structural members for the purpose of identifying and classification of damages/distresses. While it is referred in connection with appraisal of concrete and embedded reinforcement that is showing some degree of distress, its application is for all buildings and structures. The rebound hammer, UPV test, Core test and half cell potential survey has been performed in fire damaged and adjoining area as shown in Fig 11-13. The results are summarized in subsequent Para.

The test results through rebound hammer at different locations of fire damaged areas have been observed around 20 MPa. The rebound hammer test on fire damaged concrete surface always gives higher results due to conversion of $Ca(OH)_2$ into carbonates resulting into hardened concrete surface. The UPV test results were found unsatisfactory (doubtful) specifically in fire damaged locations due to presence of excessive pores caused by fire. The UPV tests results in unaffected areas were found satisfactory to good.

The core results obtained from extracting cores from fire exposed columns and unaffected columns showed compressive strength between 14.0 – 29.0 MPa, whereas the same for slab has been observed as 15-27 MPa. The densities of cores were also found very low varying between 2140 to 2400 kg/m^3. The detailed inspection of cores revealed presence of excessive pores & honeycombing due to fire exposure. Water absorption of cores was found more than 2.5-6.5% which further corroborates the loss of water due to fire exposure.

The Corrosion analyzing has been also carried out to assess the potential regions of corroding reinforcement non-destructively. It has been observed that probability of corrosion is varying from 5% to 50%.

Inspection of reinforcement in exposed beams due to fire revealed that the damages to reinforcement were not of very serious nature. However, the damages of structural reinforcement in particular slabs are found very serious. Further, it has also been observed that the structural reinforcement in above slabs are warped, buckled and distorted due to fire. The reinforcement has lost its complete strength at a few locations in slabs. The fire exposed steel samples are collected from roof slab from critical locations and tested as per IS: 1608 and ISO: 6892 which is presented in Table 1. The reinforcement in columns is observed virtually no damages because it was not exposed to elevated temperature.

Table 1: Tensile Strength of Fire Damaged Reinforcing Bar

Sl No.	Actual Dia of bar (mm)	Average Measured Dia of Bar (mm)	% elongation	Ultimate strength (MPa)
1	8	7.56	9.25	410
2	8	7.42	8.50	390
3	8	7.80	11.82	438
4	10	9.79	14.80	428

The overall conclusion drawn from the visual inspection and detailed investigations is that the concrete after fire is disintegrated has become porous due to loss of water causing cracking, severe spalling, honeycombing, bulging, surface crazing etc. The fire exposure to reinforcement of slabs and beams has caused distortion and deflection in slab/ beam, delamination. Therefore, appropriate rehabilitation/strengthening of structural members is required for the region which is exposed to fire and elevated temperature.

Figure 11. Rebound hammer testing of beams on fire damaged portion

Figure 12. Ultra-sonic pulse velocity testing of fire damaged beam

Figure 13. View of honeycombed cores with fractured plane

Material Characterization and Chemical Analysis

Behaviour of a concrete structural members exposed to fire is dependent on the composition and characteristics of concrete. At elevated temperatures or accidental fire, concrete surface exposed to heat significantly affects internal stress and thus cracks of varying sizes are generated owing to the heterogeneous volume dilation of ingredients and built up pressure in the pores. There are also some chemical and micro - structural changes in the cement concrete fire exposed such as migration of water, increase in dehydration and thermal incompatibility of the interface between cement paste and aggregates.

The physicochemical changes in the cement paste of concrete after fire are analyzed by XRF, FE-SEM, TGA/DTA and XRD techniques (Fig. 14-16) along with the chemical analysis.

The pH values of concrete sample upto depth of 30 mm from concrete surface exposed to fire are found excessively low (around 9.0) as compared to inner concrete sample (around 12.0). The chloride and sulphate content in concrete sample has been found within permissible limit.

XRF results indicate that silica content in the samples increases while CaO content along with other minor constituents decreases with the extent of severity of fire. As per specified value, CaO content in the fired sample is at minimum level indicating dissociation of C-S-H phases which is responsible for loss of strength. Further, TGA/DTA analysis indicate that the samples exhibit 10-12% ignited loss (water & other volatiles) on heating from 24 to 1000^0C which is very low as per the reported value for fully hydrated paste of typical cement. It is reported that for typical Portland cement, the ignited weight loss is 42 - 44%. The recorded Ca/Si ratios vary irregularly from point to point as viewed in FE-SEM and found to be reduced as compared to the reported 1.5 to 2.0 of typical OPC. The difference in the microstructures in the form of phases observed under FE-SEM is also supportive of changes in the physiochemical properties of concrete under accidental fire.

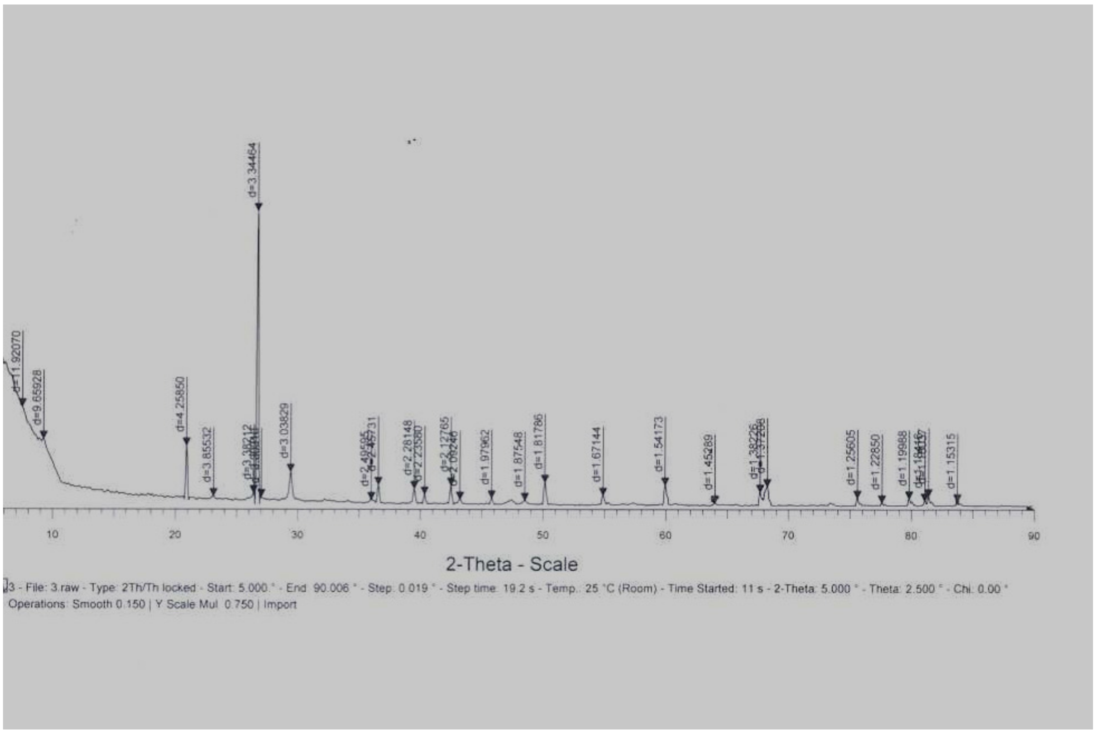


Figure 14. X-Ray diffraction patterns of fire damaged concrete samples - high fire intensity

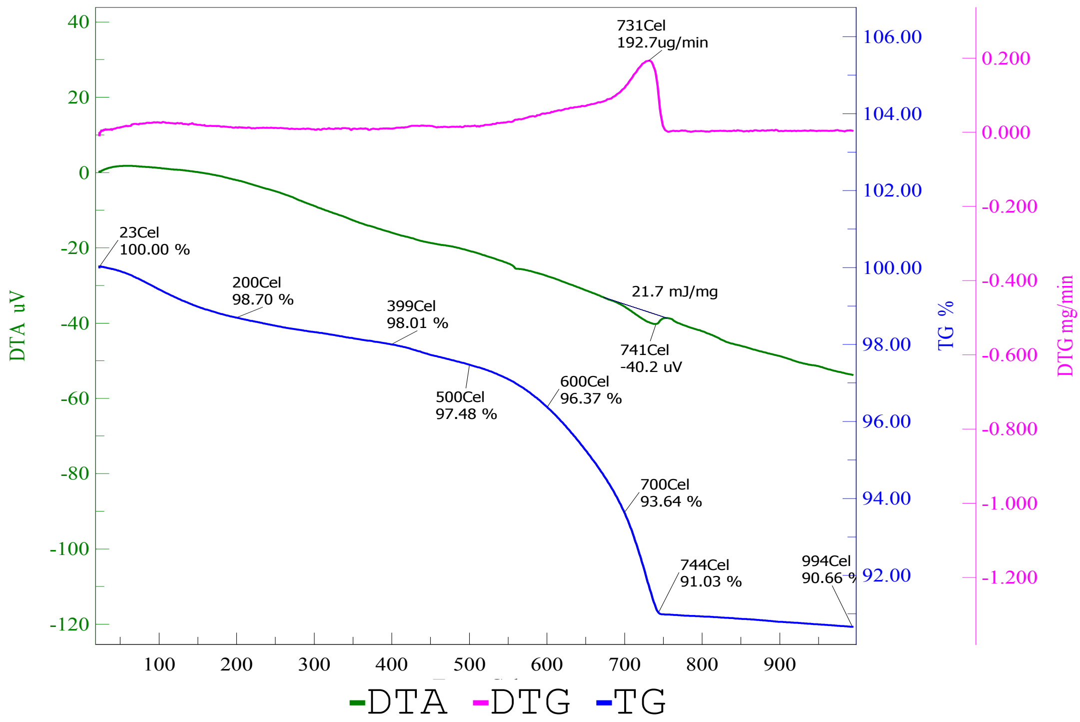
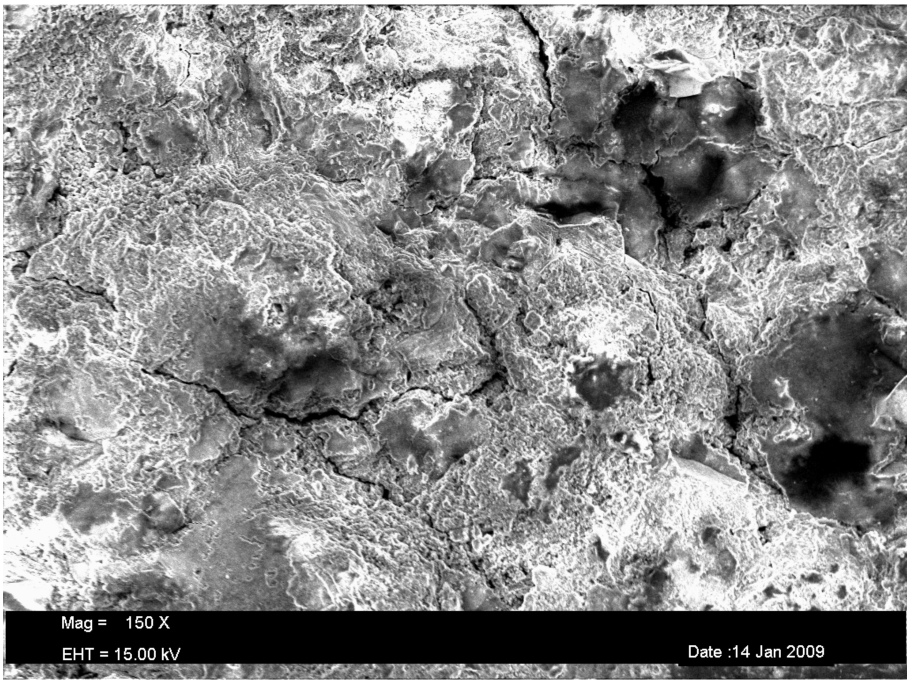


Figure 15. TGA/DTA of fire damaged concrete- high fire intensity



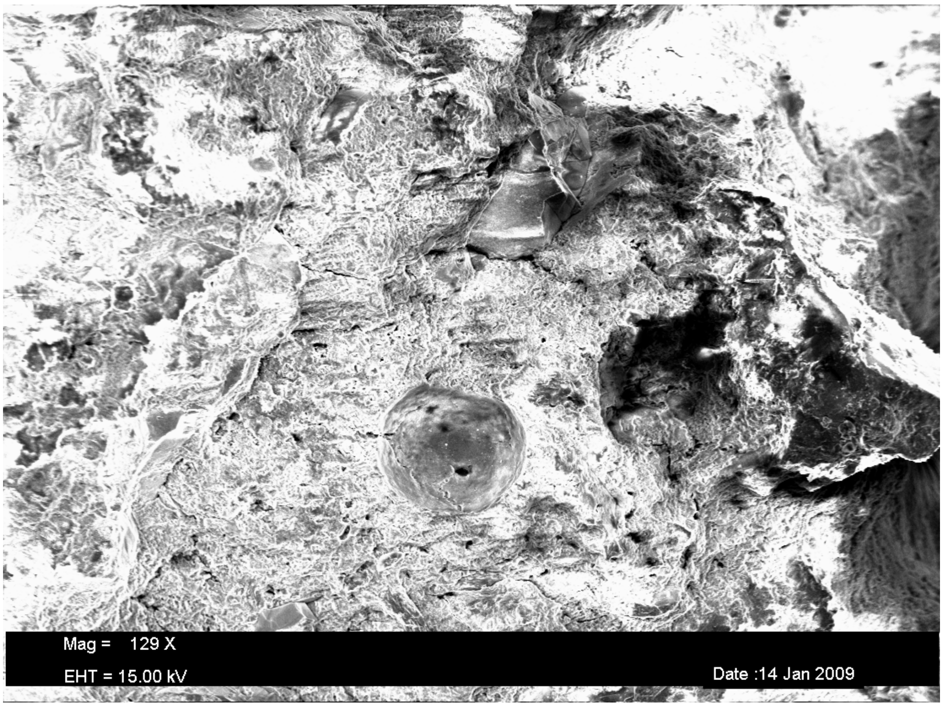


Figure 16. SEM micrographs of fired concrete samples

Damages Classifications

Based on the information collected during visual inspections indicating the condition of surface and structural elements such as spalling, exposure and condition of reinforcement, cracks, deflections, distortion, honeycombing etc.; studying of drawings, design calculations, discussions; co-relating with NDT evaluations & laboratory tests, the structural members have been designated with various damage classifications as given in Table 2 . Further, Table 3 provides type of repair to corresponding designated damages.

Table 2. Damage Classification of Structural Members

Class	Classification	Description
1	Cosmetic damage	Soot deposits and discoloration; Concrete not damaged except for insignificant crazing or spalling of plaster or finishes
2	Technical surface damage	Soot deposits, some crazing occur, Surface covering (render, plaster, painting) locally damaged. Local spalling only of concrete if any without structural reinforcement being exposed. Concrete discoloured. UPV Test (good), Rebound Indices satisfactory.
3	Structural damage surface	Soot –deposit, Surface coverings usually totally damaged. Considerable spalling along and around the edges, leaving reinforcement partly exposed, However bond between concrete and reinforcing bars intact. Concrete discoloured (buff). Buckling/ distortion were normally not noticeable.
4	Structural damage cross-section (interior)	Spalling and cracks all around the member with major parts of reinforcement exposed. Bond between concrete and reinforcement bars can be locally destroyed. Minor distortion noticeable. The stability of member not seriously affected.
5	Structural damage to member	Extensive spalling leaving significantly reduced cross section. Almost all of the reinforcement bars exposed, bond between bars and concrete is broken extensively. Similar but noticeable deformations/ buckling/ deformation do occur indicating reduced load bearing capacity

Table 3. Class of Damage Vs Type of Repair

Class of Damage	Type of Repair	Description
1	Superficial repairs	Cement plaster application over treated surface, finishes, painting etc
2	Minor structural repairs	polymer modified cement sand mortar, epoxy mortar treatment etc.
3	Principal repairs	Sealing of honeycombed portion by cement grouting, sealing of structural cracks by epoxy injection grouting, ferrocement lining, shotcreting by providing welded wire mesh, micro-concreting etc.
4	Major Structural Repair	Jacketing with concrete/ micro concrete with additional reinforcement, FRP wrapping, sprayed concrete with additional reinforcement etc.
5	Demolish and Re-casting	Demolishing the heavily damaged members and recasting with fresh concrete with additional reinforcement.

Ambient Vibration Measurements

Based on the available information and structural drawings, 3D model of the building portion has been created using STADD Pro software and its free vibration analysis has been carried out. The analytical frequency obtained is 1.49 Hz. This clearly indicates that there is appreciable shift in natural frequency of the building from the analytical frequency, to the tune of nearly 20%. While analyzing first six modes of the model, it is seen that at higher modes i.e. in 4^{th}, 5^{th}, and 6^{th} mode, the fourth and fifth floor of the building is subjected to higher amplitude. Further, it was noted that the frequency of the upper story where fire took place has been shifted on lower side, attributing the fact that fire has caused considerable degradation in material properties making fourth and fifth story of the building vulnerable, hence needs immediate strengthening.

Rehabilitation Measures

- Based on the findings of inspection and interpretation of test results, repairs and structural strengthening measures which include polymer modified cementitious/epoxy grouting, sealing of cracks, synthetic fibre based guniting, micro concreting with additional reinforcement, FRP wrapping of structural members with protections and demolition and re-casting of slabs are carried out. Further, sand blasting has been carried out for removal of loose/ porous concrete, soot and traces of organic matters from surface of the structural members.
- The excessive overburden dead load on 5^{th} floor has been removed to reduce the load effects on fire damaged structural members.
- The fire protection measures are also implemented to improve fire safety level of fire damaged area.

CONCLUSIONS

After fire, a through appraisal of building is necessary to suggest appropriate rehabilitation system. An estimate of fire severity in terms of fire exposure of an equivalent standard test has been required. Based on visual examinations accompanied with series of tests, characterization of materials and damage assessment, rehabilitation & strengthening measures can be drawn up properly. Normally concrete exposed to temperature above 300^{o}C is replaced if possible. Specialist structural and materials engineers can assess fire damaged structures using a range of forensic engineering techniques and specify well informed repair solutions.

ACKNOWLEDGEMENT

The author thanks the Director, CSIR-Central Building Research Institute, Roorkee for grant of permission for this presentation. The contributions of various Scientists and S&T staff in these investigation works and sponsors are also acknowledged.

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S. K. Singh

Mr. S. K. Singh is presently Principal Scientist in the Structural Engineering Group and Associate Professor, Academy of Scientific Innovative Research (AcSIR) at CSIR-Central Building Research Institute, Roorkee. He is also Vice President (North), Indian Concrete Institute. He has rendered his significant R&D contribution to CSIR-Structural Engineering Research Centre during 1996-2000 and engineering services as Manger (Technical in National Highways Authority of India (NHAI) from 2000 to 2005. Presently, he is continuing his R&D and academic contribution to CSIR-CBRI Roorkee.

His main research interests are in the areas of *Distress Diagnosis and Rehabilitation of Structures, Post Fire Investigations, Development and Performance Evaluation of New Construction Materials, , Earthquake Resistant Design & Construction, Quality Assurance and Quality Audit, etc*,. He is deeply associated with more than 75 Sponsored Research & Consultancy projects and published more than 65 research papers in International & National Journals/ Conferences. Further, he is continuously providing technical service to many government institutions and private organizations with regard to repair & rehabilitation, design and vetting of structures, construction, planning, quality assurance of civil engineering projects etc.. He is deeply involved in rehabilitation of several important buildings and bridges. He has also published two books on *"Seismic Resistant Design & Rehabilitation of Structures" & "Earthquake Resistant Design & Construction Practices" and* edited two proceedings namely "Role of infrastructure for Sustainable Development" and "Innovations in Repair and Rehabilitation of Structures". He has supervised several M. Tech. dissertations.

Singh is the recipient of the prestigious *Young Scientist Award* by Council of Science & Technology, Govt. of UP for the year 2000. Singh is an active member of BIS, IRC & other Committees. He is member of Central Monitoring Cell, Monitoring of Integrated Housing & Infrastructure Projects for Urban Poor under Jawaharlal Nehru Urban Renewal Mission (JNNRUM). He is member of of CIB, Netherlands. He is also member of building committees of several organizations such as HNBG Central University etc. He is Fellow of Indian Concrete Institute and The Institution of Engineers (India) and life member IRC, ISWE, ISCMS, ACI, ISET, & IGS. He has served as Honorary Secretary of The Institution of Engineers (India), Roorkee Centre for the Year 2008-10. He has also served as founding Coordinator and Honorary Secretary of Ghaziabad Chapter of Indian Concrete Institute from 1997 to 2002. He also served as member of National Executive Committee of ISET, ISCMS, and other professional bodies. He has organized several national and international workshops, seminars, and training programmes all over the country and delivered Key Note Lectures in many national and international conferences.

PRACTICAL USAGE & ADVANTAGES OF HOT DIP GALVANIZED REBAR IN REINFORCED CONCRETE STRUCTURES FOR DURABILITY

D. S. Joshi and Neelkanth D. Joshi

Abstract: The importance of use of galvanized rebar to protect the rebar against corrosion specially in coastal areas and thereby improving longevity of reinforced concrete structures is brought out in his article.

Keywords : Reinforced Concrete Structures, Coastal Areas, Carbonation, Chloride Attack, Durability Of Concrete Structures, Hot Dip Galvanization Of Rebar, Zinc Coating.

Introduction:

Fundamentally a durable reinforced concrete structure cannot be created in the absence of well designed, good quality, compact, strong, impermeable and durable concrete with adequate compact cover to reinforcement.

It is also important to protect the steel reinforcement against corrosion as concrete loses its capacity to passivate due to carbonation, chlorination and attack of other chemicals.

Available Option For Protecting Steel

- Cement Polymer Composite Coating (CPCC).
- Epoxy Coating – Brush Application.
- Fusion Bonded Epoxy Coating.
- Galvanizing.
- Use of Stainless Steel Reinforcement.

It was learnt from few corners that hot dip galvanized rebar were not used in many countries like USA, Japan and other European countries. Therefore, a systematic study of international codes, Indian codes, and websites was made.

The Essential Requirements of A Good Coating To Rebar Embedded In Concrete Are -

Adequate bonding to steel and concrete
Free from localized defects
Good abrasion resistance
Good bend ability
Good impact resistance
Alkali and chloride resistance
No adverse effect on concrete or steel
Ease of patch repair
Ease of application
Economical

Galvanizing produces a damage resistant metallurgical bond and creates layers of alloys of zinc and steel which are harder than steel itself (Figure 1.). Theses layers protect the steel and offer excellent resistance to abrasion. Galvanizing protects the steel from corrosion, increases the damage resistance of the reinforcement. Since, galvanizing covers all surface areas the fabricated rebar is completely protected regardless of any other protection.

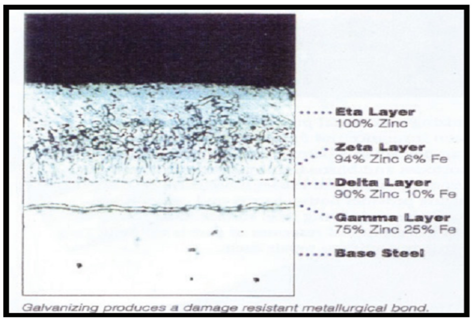

Fig. 1.

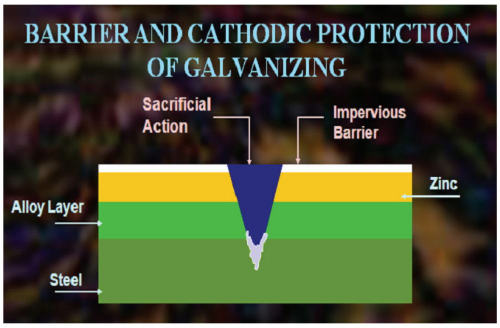

Fig. 2.

Zinc coating has a unique property which covers up any holidays / scratches that may occur in the coating. This is due to the electrochemical property of zinc which is more electro positive than steel and has a tendency to donate electrons to exposed steel. Hence if a holiday occurs in the zinc coating the zinc from surrounding area comes and covers it up as shown in Figure 2.

The pH of cement which varies during the first four hours is very important while considering the barrier protection of zinc coatings. pH depends on large degree on the alkali content of the cement but even for cement with an alkali content that could be regarded as exceptionally high and therefore unusual, the pH value does not exceed about 13. Although, sometimes under very exceptional circumstances it does reach 13.5 to 13.8. In normal concrete which has ph of 12.4 to 12.5, the corrosion rate of zinc is at minimum. Zinc is most protective when the ph of the surrounding atmosphere does not exceed the range 5.5 to 12.5. However, factors such as aeration, temperature, polarization and presence of inhibiting ions may have considerable influence on corrosion.

The reaction between Zinc and Calcium Hydroxide results in the formation of Calcium Hydroxy Zincates that protects the zinc layer. It also helps developing good bond between concrete and steel.

Pozzolonic additives such as blast furnace slag, fly ash and silica fumes in Portland cement considerably improve the performance of galvanized rebar in concrete as they reduce porosity, heat of hydration and help control pH of concrete.

Carbonation

Galvanized reinforcement is good for resisting the effects of the carbonation because of the much wider range of pH (up to 8) over which the zinc coating remains passivity. Normal steel typically depassivit when the pH of concrete drops below about 11.5. It is therefore apparent that as the carbonation front moves past a galvanized rebar, little or no effect will occur until the concrete adjacent to the reinforcement is almost completely neutralized. Relative depth of carbonation with respect to that of 50% of relative humidity is shown in Figure 3, where D is carbonation depth at 50% relative humidity.

Carbonation Tolerance

Though Zinc can be depassivated and attacked in the presence of Chloride ions, the tolerance of galvanized reinforcement to Chloride depassivation is substantially higher than that of untreated steel. If Chloride level in concrete is in the range of 0.2 to 0.3% by weight of cement it leads to sever corrosion of untreated steel. The susceptibility of concrete structures to the intrusion of Chloride is the primary incentive for use of Galvanized steel reinforcement. Galvanized reinforcing steel can withstand exposure to Chloride ion concentrations several times higher (at least 4 to 5 times) than what causes corrosion in black steel reinforcement.

Black steel in concrete typically depassivates below a Ph of 10, Galvanized reinforcement can remain passivated at a lower Ph (Ph = 5.5) there by offering substantial protection against the effect of carbonation of concrete. A concrete chloride level of 1.1 to 1.3 pounds of chloride per cubic yard of concrete is the threshold level at which corrosion of bare steel will occur. The galvanized rebar performs well due to presence of the galvanized coating despite chloride levels in some instances between 9 and 10 times the threshold corrosion level of rebar. The comparative extension of service life of reinforced concrete structure with galvanized and non galvanized rebar is shown in Figure 4.

Galvanizing Of High Strength Steels -

The problems related to galvanizing of high strength steel such as risk of cracking due to Hydrogen embrittelement are not likely to be encountered in Fe 415 or Fe 500 category steel reinforcement. The galvanizing should be carried out as recommended in I.S. 12594 - 1998 for Hot Dip Zinc Coating On Structural Steel Bars For Concrete Reinforcements and the steel should be finally Chromate passivated with 0.2% solution of Potassium Dichromate.

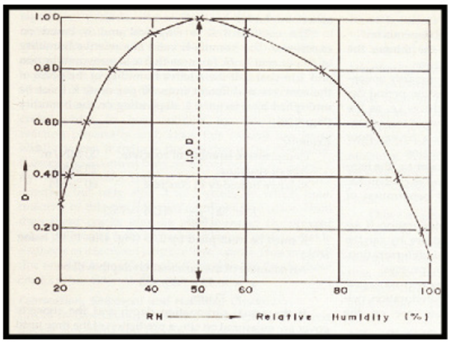

Fig. 3.

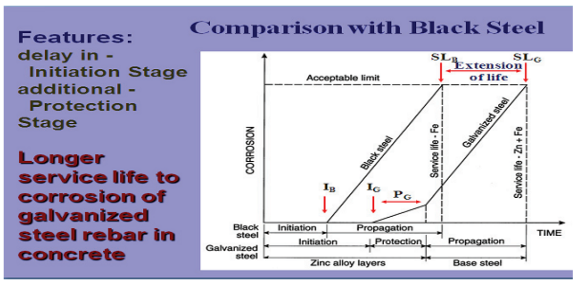

Fig. 4.

Design Criterion

When galvanized steel is specified the design requirements and installation procedure employed should be no less stringent than for structures where non-galvanized reinforcement is used.

Steel Selection

The concrete reinforcing steel to be galvanized shall confirm to standard I.S. specifications with desirable mechanical properties of steel as specified in I.S. 1786 from view point of ductility requirements in earthquake resistant reinforced concrete structures. Hot dip galvanized reinforcement should preferably be used from view point of durability in the structures especially in coastal area and offshore structures susceptible to chloride attack.

Detailing Of Reinforcement

Detailing of galvanizing reinforcing steel should confirm to the design specifications for non-galvanized steel bars and to normal standard practice. Care shall be taken to see that, non galvanized bars or binding wires do not come in contact with galvanized bars to avoid bimetallic reaction.

Bending Of Bars

Hooks or bends should be smooth and not sharp. Cold bending should be in accordance with the 4 standard recommendations for normal steel. It is advisable to bend the bars after galvanizing whenever possible. When galvanizing is performed before bending, sometimes cracking and flaking of the galvanized coating at the bend may occur but is not a cause for rejection. The tendency for cracking of galvanized coating increases with bar diameter and with severity and rate of bending.

Storage And Handling

Galvanized bars can be stored outdoors with complete assurance. This ease of storage makes it feasible to store standard lengths so that they are available on demand. Galvanized steel can be handled and placed in the same manner as normal steel reinforcement because of the great abrasion resistance of galvanized steel.

Welding

Welding of galvanized reinforcement poses no problems provided adequate precautions are taken. These include a slower welding rate and proper ventilation. The ventilation which is normally required for welding operations of normal high strength deformed steel bars is considered adequate. The heat damaged areas need to be repaired.

Local Repairs Of Coating

Local removal of galvanizing coating in the area of welds, bends or sheared ends should be coated with liquid Zinc Substrate containing 92% Zinc Sulphate with other bonding agents, as per standard practice for repairs of damaged and uncoated areas of Hot Dip Galvanized coatings.

Thickness Of Coating

The life of the galvanized coating depends on coating thickness and the severity of the exposure. The required thickness of the zinc coating must be sufficient to form a protective layer and also to allow a margin for corrosive attack when the concrete environment surrounding the steel has lost its alkalinity through carbonation, sulphation or chloride penetration.

A coating weight of 610 gms/sq.m (86 micron thickness) readily achieved by Hot Dip Galvanizing Process is suitable and gives adequate protection in concrete and during short term storage on site. However thickness in excess of 100 microns is achieved in practice on steel of smaller diameters.

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Dinkerrai S. Joshi

Shri Dinkerrai S. Joshi, is a Consulting Structural Engineer, M/s. JOSHI CONSULTANTS, Mumbai. He completed B. E (Gold Medallist) and M.E (I^{st} Class) in Structures from the University of Poona. He has 9 years of teaching & research at Sardar Patel College of Engineering (Mumbai). Shri D. S. Joshi is associated with academic activates of various Universities in the State of Maharashtra and Gujarat, being the examiner for graduate and post-graduate courses in Structural Engineering. He has been practicing as a Consulting Structural Engineer for the past 46 years and has achieved a place of eminence in the field. He has designed number of structures including residential high rise buildings, commercial buildings, industrial structures of various heights, spans and loading for various products, educational institutes, places of worship, theatres, auditoria, water storage reservoirs, retaining walls, various types of roads in different terrains and bridges. He has designed more than 9000 structures in his carrier in India & Middle East which include huge individual projects covering more than 100 acres of land. He is often appointed as the "Court Commissioner". One of his notable contribution is unique earthquake resistant design and high speed construction technique using R.C.C. cast in situ hollow wall construction. He is also member of professional bodies such as ISTE, ACI, ASTR, TRMA, IZDA. He is keen in disseminating the knowledge gained, through lectures/seminars/courses/publishing/ journals/books. He published a book on "Design of Reinforced concrete Structures For Earthquake Resistance". The findings of his work on "Photo-Elastic Investigation Of Effect Of L / D Ratio On Portal Frame – M.E. Thesis 1965" have found place in IS – 456 : 1978 in Section 4, in a clause on deep beams, and is repeated in I.S. 456 – 2000. He has conducted many seminars/courses on "Earthquake Resistant Design Of Structures". Shri D. S. Joshi helps the Society members in the redevelopment of their buildings right from selection of the developer, to completion of project till occupation by acting as an overall consultant.

ANALYTICAL APPROACH TO RETROFITTING OF HERITAGE MASONRY DOMES

Mahesh Varma[1] and Siddhartha Ghosh[2]
[1] Nandadeep Designers and Valuers Pvt. Ltd, Aurangabad
[2] Department of Civil Engineering, IIT Bombay

Abstract
The assessment of the heritage masonry domes for retrofitting purpose requires the appropriate method of structural analysis. Specifically in absence of reliable data for mechanical properties of masonry, it poses the greater challenges to the engineers. This paper demonstrate the development of appropriate method of structural analysis of these heritage masonry domes, finite element thrust line approach. The proposed method considers the effect of hoop compression forces on the thrust line of masonry dome, and allows the release of hoop tension forces. The presented iterative analysis method gives better approximation for extended hoop tension region and so it gives better understanding of structural stability of cracked masonry dome.

Keywords: Masonry, Dome, Thrust line, Finite element

INTRODUCTION

Masonry domes are a common feature for historical monuments covering arge spans. Pantheon (Italy), St. Peter's basilica (Italy), Hagia Sophia (Turkey), and Taj Mahal (India) are few examples of the wonders created by ancient builders over the past two thousand years that include large masonry domes. The retrofitting of these structures require good understanding of it's structural behavior. But the non-availability of mechanical properties data for such structures proves to be a major hurdle in it's structural analysis. Can the stability of these structures be studied without fair judgment of mechanical properties of these structures? This question distinct the problem of masonry dome analysis from other structural analysis problems. The usual structural theory of framed, trusses or shell structures made of reinforced concrete or steel is of no use to study masonry architecture [11].

REVIEW OF AVAILABLE METHODS FOR HERITAGE MASONRY DOMES

In ancient times, the stability of such structures is established by ancient builders using understanding of simple geometrical rules. Later with the intent of the thrust-line method more precise understanding for arch stability is generated. Heyman [7] has formally documented thrust line approach and extended the use of thrust line method to the dome stability study. The need of values for mechanical properties was completely eliminated from such thrust line method and it is understood to be the major advantage of this method. It is well understood that such stone and block masonry structures generally fail due to their instability and not due to the lack of strength of the material [7, 5, 1, 19] as the stresses level remain much lower than that of permissible stress for masonry. Hence the stress outputs of the present form of linear finite element method remain useless of such heritage structure analysis. The present form of graphical thrust line method has many limitations and is not as versatile as finite element method. Hence due to its wide applicability, finite element method remains the method of preference for many researchers. A review of available methods for the analysis of masonry domes reveals that these methods can be broadly categorized as, a) thrust line approach and b) finite element-based approach.

Thrust line approach

The graphical thrust line approach to masonry arches has been the traditional method of choice for architects and engineers for many centuries. Heyman [7, 10, 8, 9] carried out extensive work on demonstrating the effectiveness of simple thrust line calculations for masonry structures. He demonstrated that stresses in masonry structures under usual loadings are insignificant and commented [7], *The question of strength of the masonry can be expected to hardly enter the calculations. Instead it is likely that individual structural elements, and the structure as a whole, must be proportioned on the basis of the stability.* This key idea of checking the stability has set the importance of the thrust line method in masonry structural analysis. The thrust line approach has been used thereafter by many researchers such as Clemente, ODwyer, Block and Zessin [5, 14, 1, and 19]. The static equilibrium equations based on assumptions proposed by Heyman [7] eliminate the estimation of mechanical properties from thrust line calculations which rely only on the geometry and the loading distribution. The graphical thrust line method based onHeyman's assumptions is simple to implement and understand, but this method is far from being versatile to handle the complex geometry, boundary conditions and redundancy of actual stone masonry structures. Conventionally, thrust line approaches idealize a masonry dome as a series of lunes ('orange slices'), as described by Poleni [8] and allows a simplified analysis of a dome, ignoring the effect of hoop forces. This approach was employed by Bouguer, Frezier, Poleni [2, 6, 17] and many others for analysis of masonry domes. Heyman [8] demonstrated the understanding that this orange slice technique ignores the contribution of hoop forces on a dome thrust line, resulting in a very conservative design for the dome thickness. Wolf [18] method for dome analysis was capable to consider the effect of hoop forces on dome thrust line, and due to consideration of resistance of masonry to hoop compression as well for hoop tension it has resulted in overlapping the thrust line over dome's meridional surface. Heyman [8] and Zssin [19] determined the minimum thickness to radius ratio, t/R, of segmental domes by assuming that hoop forces exist only in the uppermost cap of the dome and did not considered the resistance of masonry to hoop tension forces.

Finite element based methods for masonry structures

Finite element method has remained dislike of many researchers for masonry analysis. Huerta [11] emphasized that finite element and discrete element method can only add "noise" to the analysis. He also expressed that use of FEM elastic analysis of masonry architecture is a purely academic exercise. Block [1] expressed similar opinion regarding the use of finite element based analysis method for the masonry structures. The major concern express by Huerta [11] and Block [1] in using _nite element method for masonry analysis is the result's sensitivity to the change in mechanical properties of the material. But with all this criticism, the enormous possibilities to model different intricacy related to mathematical modeling of masonry have remained the major attractions for the researchers.

Masonry finite element modeling can be categorized as micro-modeling and macro modeling [21]. The strategy of micro modeling can be traced back to Page [15]. He considered elastic bricks and inelastic mortar for finite element modeling. In micro-modeling, the failure is considered to occur at the interface if the tensile shear bond strength criterion is violated.
The major difficulty in using this approach is in the intricacy in predicting mortar joint positions or their thicknesses in masonry. Buhan [3] pointed to the difficulties with practicing this approach due to numerical difficulties with increased size of the problem. Milani [12] pointed that *"a drawback of this approach is related to the necessity of modeling separately units and mortar, so limiting its applicability to small panels."*

This drawback of the micro modeling approach leads to a need for homogenized macro modeling approach considering the masonry as a homogeneous material. However, constitutive laws for such homogenous equivalent masonry remain different for different masonry patterns. The homogenization approach is used by researchers to predict the stress value, and the safety of the structure is judged by the strength criterion. The limitations of using this homogenization approach are:

1. The type of texture i.e. the way in which the blocks and mortar is arranged, deeply influences the masonry mechanical response [4]. This texture is in masonry structures is not usually known, which leads to difficulties in obtaining generalized homogenized mechanical properties for the masonry.

2. The homogenization approach is used mostly to predict the failure, based on a stress based criterion. Stress results are sensitive to mechanical properties [16], and with slight variation due to site conditions, the assumed mechanical properties could lead to incorrect predictions of failure.

3. The mechanical properties required by model are derived from experimental data and the results are limited to the loading conditions under which the data are obtained. [12, 13, 21]

4. The stability of the structure is not represented in simple terms as it is indicated in the thrust line method [1]

There are many limitations in applicability of finite element method of analysis to masonry structures as stated above, but still, finite element method is the versatile method to handle complex geometry, boundary conditions and redundancy of actual stone masonry structures which is seldom possible in other methods of analysis.

STRUCTURAL ISSUES WITH MASONRY DOMES

The hoop tension cracks in the heritage masonry domes are very common. As shown in Photo 1 of Bibi-Ka-Maqbara (Aurangabad, India), the evidences of meridional crackings are apparent from the clamps fixed on the outer marble cladding. The cracks are found extended in the terrace level floor and on the supporting walls. Similar cracks can be seen on the inner face of the dome in case of Gol-Gumbaz (Bijapur, India). This dome was repaired in 1936 by adding a reinforced concrete to the outside to help tie the cracked segments of the dome together. And hence the external face cracks are not visible in case of Gol-Gumbaz. Gol-Gumbaz also has the extended cracks on the terrace level oor and in the supporting walls. In Europe, for example the major domical structures in Italy – Pantheon and St. Peter cathedral has also evidences for such meridional cracks due to the existence of hoop tension forces. As shown in Photo 2 of St. Peter cathedral, the ancient builders have tried to arrest these cracks with buttery wedge stones. Obviously these small buttery wedges were not sufficient to resist the huge amount of the hoop tension forces generated in this massive span dome. This also shows the inability of the ancient builders to quantify the hoop tension forces in masonry domes. These domes still remain stable with these hoop tension cracks, as it is mentioned by many eminent researchers, the domes stand as series of arches with the common key stone (the upper cap of dome, in hoop compression). But such cracks certainly affect the stability of the masonry dome and it is essential to know the effect of these tension cracks on the stability of masonry domes.

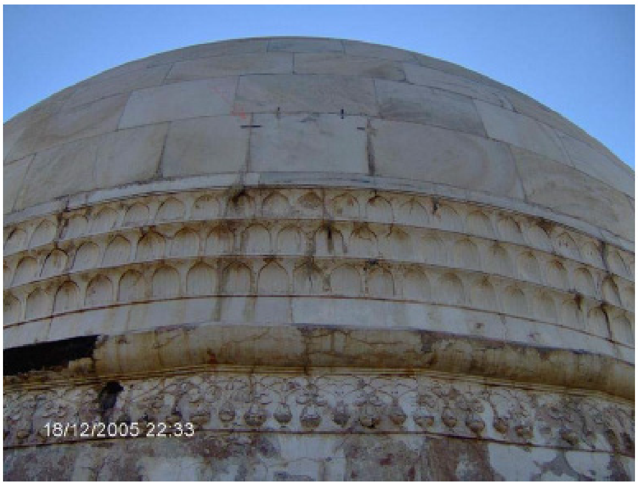

Figure 1: Evidences of meridional cracks in Bibi-Ka-Maqbara

Considering the pros and cons of both the approaches, the need of a method combining the simplicity of the thrust line approach and the versatility of the finite element method is apparent for analysis of masonry dome like structures and it leads to the following criteria for the development of new method:

1. Masonry structures are critical for stability and not the stress. Hence stability should be primary check as in thrust line method and stress should be secondary check as considered in finite element analysis.

2. The material properties of masonry cannot be predicted with great accuracy, and it is apparent from thrust line hypothesis that the stability is almost independent of material properties for practical consideration. Hence the stability check should not be much sensitive to mechanical properties.

3. The method should be versatile to model the intricacy of actual structure as it is possible infinite element based method.

This paper further demonstrate the appropriate method of masonry dome analysis,finite element thrust line approach, to satisfy above criteria. Development of such method will assist the engineers to have better assessments for the heritage masonry dome structures and to plan the retrofitting strategies.

Figure 2: Evidences of meridional cracks in St. Peter cathedral

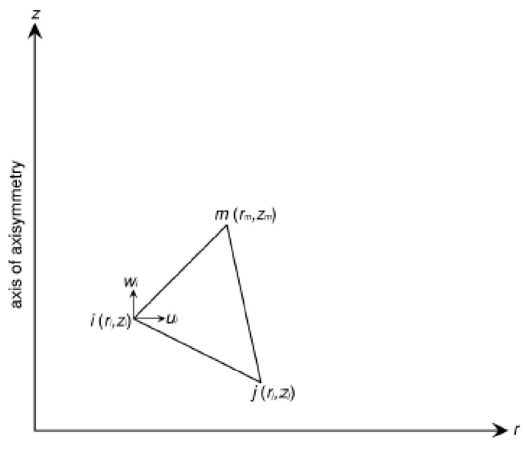

Figure 3: Axisymmetric element

DERIVATION OF SUITABLE ELEMENT FOR MASONRY DOMES

General

In the regular finite element analysis of a masonry dome, the conventional axisymmetric element considers hoop tension as well as hoop compression forces to be resisted by the stone masonry. Whereas in reality, tensile hoop forces result in the cracking of the masonry dome. In addition, the conventional axisymmetric element under-estimates the detection of the dome. This chapter demonstrates the development of suitable axisymmetric element for analysis of axisymmetric masonry dome. The material is assumed as isotropic with modulus of elasticity E and poission's ratio *v*.

The three noded axisymmetric element *i, j, m* with 2 DOF per node, in the (*r, z*) coordinate as shown in Fig. 3 is considered with z as axis of axisymmetric. The displacement vector a at any node *i* in axisymmetric

element is considered as

$$\mathrm{a_i} = \begin{Bmatrix} u_i \\ w_i \end{Bmatrix} \tag{1}$$

and the element displacement vector is

$$\mathrm{a^e} = \begin{Bmatrix} a_i \\ a_j \\ a_m \end{Bmatrix} \tag{2}$$

Development of the stress-strain relation matrix

The stress-strain relationship for the conventional axisymmetric element with isotropic material is defined as [20]:

$$\sigma = \mathrm{D} \times \epsilon \tag{3}$$

Where D is the stress-strain relationship matrix, and the strain and stress matrices are defined as:

$$\epsilon = [\epsilon_r, \epsilon_z, \gamma_{rz}, \epsilon_\theta]^T = [\frac{\partial u}{\partial r}, \frac{\partial w}{\partial z}, \frac{\partial u}{\partial z} + \frac{\partial w}{\partial r}, \frac{u}{r}]^T \tag{4}$$

$$\sigma = [\sigma_r, \sigma_z, \tau_{rz}, \sigma_\theta]^T \tag{5}$$

($\epsilon\theta$,$\sigma\theta$)are the strain and stress in hoop direction.

For this conventional axisymmetric element, stress-strain relation can be defined as:

$$\begin{bmatrix} \sigma_r \\ \sigma_z \\ \tau_{rz} \\ \sigma_\theta \end{bmatrix} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \begin{bmatrix} 1 & \frac{\nu}{1-\nu} & 0 & \frac{\nu}{1-\nu} \\ \frac{\nu}{1-\nu} & 1 & 0 & \frac{\nu}{1-\nu} \\ 0 & 0 & \frac{1-2\nu}{2(1-\nu)} & 0 \\ \frac{\nu}{1-\nu} & \frac{\nu}{1-\nu} & 0 & 1 \end{bmatrix} \begin{bmatrix} \epsilon_r \\ \epsilon_z \\ \gamma_{rz} \\ \epsilon_\theta \end{bmatrix} \quad (6)$$

Now for masonry dome analysis to be consistent with Heymans assumptions, it should be assumed that the masonry does not resist any tension. Therefore, for the element where there is tensile stress along the hoop direction, the value of this hoop tension stress should be set to zero to account for cracking; $\sigma\theta = 0$

Substituting this in Eq.6, we can write

$$\epsilon_\theta = \frac{\nu}{1-\nu}\epsilon_r + \frac{\nu}{1-\nu}\epsilon_z = \frac{\nu}{1-\nu}(\epsilon_r + \epsilon_z) \quad (7)$$

Because [E(1-v)] = [(1 + v)(1 – 2v)] cannot be zero.

Now replacing the value of $\epsilon\theta$ to Eq.6, we get

$$\sigma_r = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}\left(\epsilon_r + \frac{\nu}{1-\nu}\epsilon_z + \frac{\nu}{1-\nu} \times \frac{\nu}{\nu-1}(\epsilon_r + \epsilon_z)\right) \quad (8)$$

$$\sigma_r = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}\left(\left(1 - \frac{\nu^2}{(1-\nu)^2}\right)\epsilon_r + \left(\frac{\nu}{1-\nu} - \frac{\nu^2}{(1-\nu)^2}\right)\epsilon_z\right) \quad (9)$$

$$\sigma_z = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}\left(\frac{\nu}{1-\nu}\epsilon_r + \epsilon_z + \frac{\nu}{1-\nu} \times \frac{\nu}{\nu-1}(\epsilon_r + \epsilon_z)\right) \quad (10)$$

$$\sigma_z = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}\left(\left(\frac{\nu}{1-\nu} - \frac{\nu^2}{(1-\nu)^2}\right)\epsilon_r + \left(1 - \frac{\nu^2}{(1-\nu)^2}\right)\epsilon_z\right) \quad (11)$$

$$\begin{bmatrix} \sigma_r \\ \sigma_z \\ \tau_{rz} \end{bmatrix} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1 - \frac{\nu^2}{(1-\nu)^2}) & (\frac{\nu}{1-\nu} - \frac{\nu^2}{(1-\nu)^2}) & 0 \\ (\frac{\nu}{1-\nu} - \frac{\nu^2}{(1-\nu)^2}) & (1 - \frac{\nu^2}{(1-\nu)^2}) & 0 \\ 0 & 0 & \frac{1-2\nu}{2(1-\nu)} \end{bmatrix} \begin{bmatrix} \epsilon_r \\ \epsilon_z \\ \gamma_{rz} \end{bmatrix} \quad (12)$$

Therefore the D matrix, defining the relation between stress and strain for axisymmetric element with release of hoop tension becomes:

$$\mathbf{D} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1-\frac{\nu^2}{(1-\nu)^2}) & (\frac{\nu}{1-\nu}-\frac{\nu^2}{(1-\nu)^2}) & 0 \\ (\frac{\nu}{1-\nu}-\frac{\nu^2}{(1-\nu)^2}) & (1-\frac{\nu^2}{(1-\nu)^2}) & 0 \\ 0 & 0 & \frac{1-2\nu}{2(1-\nu)} \end{bmatrix} \quad (13)$$

Defining strain-displacement matrix B and stiffness coefficient 'Keij ' With σθ = 0, independent strain vector reduces to

$$\epsilon = [\epsilon_r, \epsilon_z, \gamma_{rz}]^T = [\frac{\partial u}{\partial r}, \frac{\partial w}{\partial z}, \frac{\partial u}{\partial z} + \frac{\partial w}{\partial r}]^T \quad (14)$$

The strain can be written in terms of element displacement vector and strain-displacement matrix B [20]

$$\epsilon = Ba^e = [B_i, B_j, B_m]a^e \quad (15)$$

With reduced strain vector of Eq. 14, the conventional element's B matrix [20] will reduce to

$$\mathbf{B_i} = \begin{bmatrix} b_i & 0 \\ 0 & c_i \\ c_i & b_i \end{bmatrix} \quad (16)$$

Where *bi* and *ci* are the function of nodal ordinate [20],

$$b_i = z_j - z_m \; ; \; c_i = r_m - r_j \quad (17)$$

Now using the general relationship to find the stiffness coefficient by taking volume integration over whole ring [20]

$$\mathbf{K^e_{ij}} = 2\pi \int B_i^T D B_j r dr dz \quad (18)$$

Here B matrix and D matrix as given in Eq. 16 and Eq. 13 are constant over the element region and hence the stiffness coefficient will reduce to

$$\mathbf{K^e_{ij}} = 2\pi B_i^T D B_j \bar{r} \Delta \quad (19)$$

Where $\bar{r}$ is the radius at centroid of the element and Δ is the area of the element. In subsequent masonry dome study, the iterative analysis algorithm is followed. For the element with hoop tension stresses the stiffness matrix coefficient of Eq. 19 is used for subsequent iteration of analysis. This procedure of replacing conventional element with new element with existence
of hoop tension stresses is continued till the convergence.

ALGORITHM FOR ITERATIVE THRUST LINE ANALYSIS

The algorithm used for iterative analysis is explained in Figure 4. The selection of element stiffness matrix coefficients is made based on the type of hoop stress (compression or tension) exists in the element. If the type of element keep changing from the conventional element to the new element due to change in element stress from hoop compression to the hoop tension, then the analysis repeats with the modified elements configuration. The analysis continues till the convergence.

DEMONSTRATION EXAMPLE

Problem description

Heyman [8] and Zessin [19] have shown through their study that the hemispherical dome will stand with the minimum thickness of 4.5% of the dome radius. This same example is considered for the demonstration and validation purpose in this section. Geometry and material properties considered for modeling are summarized in Table 1 and Table 2. Nodes at the springing level of dome are considered as restrained in both 'x' and 'y' directions.

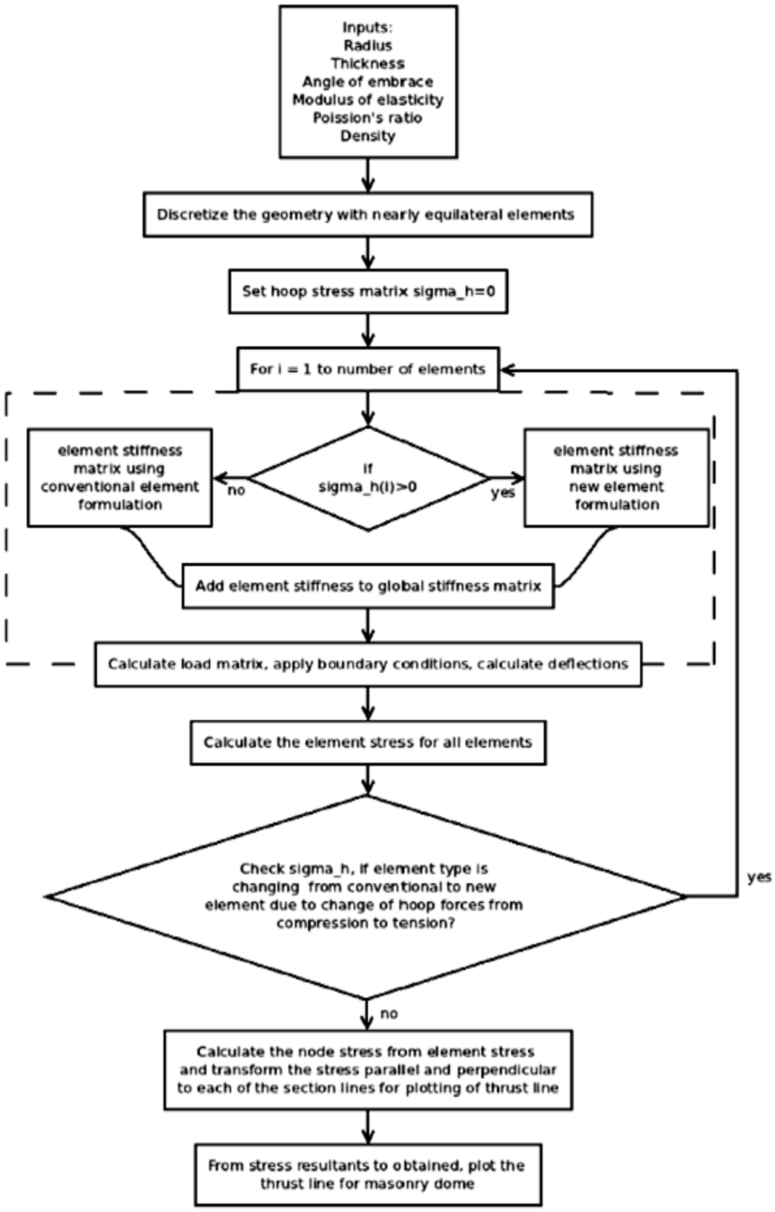

Figure 4: Algorithm for thrust line analysis of masonry dome using finite element method

Table 1: Geometry

Problem	Radius(mid surface)	Thickness
t/R=0.045	5 meter	0.225 meter

Table 2: Material Properties (assumed for analysis)

Property	Value
Modulus of elasticity	2 x $10^2 N/m^2$
Poisson's ratio	0.1
Density	2000 Kg/m^3

Results

This dome is analyzed using the new proposed method, finite element thrust line approach. The first iteration of analysis use conventional elements and it leads to the region of hoop tension as shown in Figure 5. The crown portion of this dome remains in hoop compression. In subsequent iteration of analysis, the elements in the hoop tension region are replaced by the new developed elements, as masonry does not resist the hoop tension forces. Such replacement of elements in the analysis results in the release of stiffness corresponding to hoop tension stresses. Due to this release of the stiffness, the elements adjacent to new elements change from hoop compression to hoop tension stress. The axisymmetric section of the dome with increased region of hoop tension in second iteration of analysis is shown in Figure 6. Continuous replacement by new elements in subsequent analysis cycle for the element in hoop tension leads to extension of hoop tension zone. As such this extension of hoop tension zone can be seen as the extension of the meridional crack due to the inability of masonry to resist the hoop tension forces. At certain analysis cycle the hoop tension region converges as shown in iteration 7 of analysis in Figure 7. It is well known fact that the thrust line location depends on the hoop forces in masonry dome. The thrust line plotted without releasing of stiffness corresponding to hoop tension almost coincides with the median plane of the dome as shown with dotted line in Figure 8. In subsequent analysis cycle, wherein the stiffness corresponding to the initial hoop tension region as shown in Figure 5 is released, the thrust line deviates from the median plane of the dome as shown with dashed line in Fig. 8. This thrust line corresponds to the thrust line plotted by Heyman [8] and Zessin [19]. Heyman [8] and Zessin [19] have considered the effect of hoop compression forces in the upper cap of the dome and released the effect of hoop tension forces from the thrust line analysis. But such release of hoop tension forces would result in increased region for hoop tension zone. The further deviation of thrust line due to effect of increased hoop tension region was difficult to consider in the graphical form of thrust line method for dome analysis. In the presented method, finite element thrust line approach the replacement of the element from conventional to new proposed element is continued till the convergence. It is observed that the thrust line deviates further from median plane as shown with full line in Figure 8. The zoom views of critical locations for thrust line are shown in Figure 9 and Figure 10. The dome is found to be just unsafe in using finite element thrust line approach, as the thrust line does not contained in the thickness of the masonry.

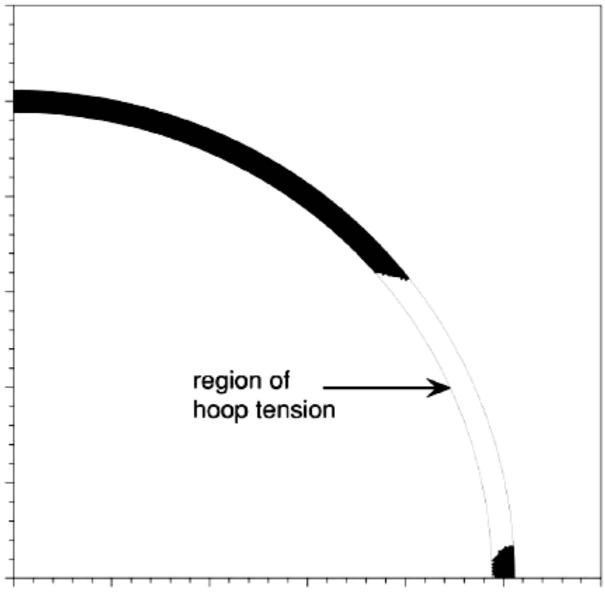

Figure 5: First iteration of analysis, result showing hoop compression and hoop tension region

This analysis include two unique significant effects on location of thrust line for masonry dome.

1. The effect of hoop compression and the effect of meridional cracks due to the presence of hoop tension forces on the thrust line location as it is considered by Zessin [19].

2. The analysis also considered the effect of extension of these meridional crack on location of thrust line due to inability of masonry to resist the tension

Many analysis are carried for given radius to thickness ratio with different radius of the dome. The qualitative thrust line diagram and the conclusion for the stability remained unchanged. This shows that the simple rules for minimum thickness to radius of the dome can easily be developed using this developed methodology of masonry dome analysis. The plotted thrust line using new element is not sensitive to mechanical properties of the material, and same is ascertain by carrying many analysis with relevant spectrum of the mechanical properties of the material. Hence this method can be used very effectively for the heritage masonry structure's analysis.

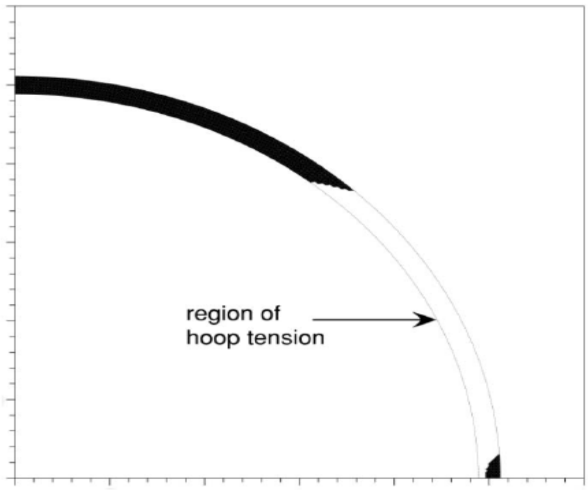

Figure 6: Second iteration of analysis, result showing hoop compression and hoop tension region

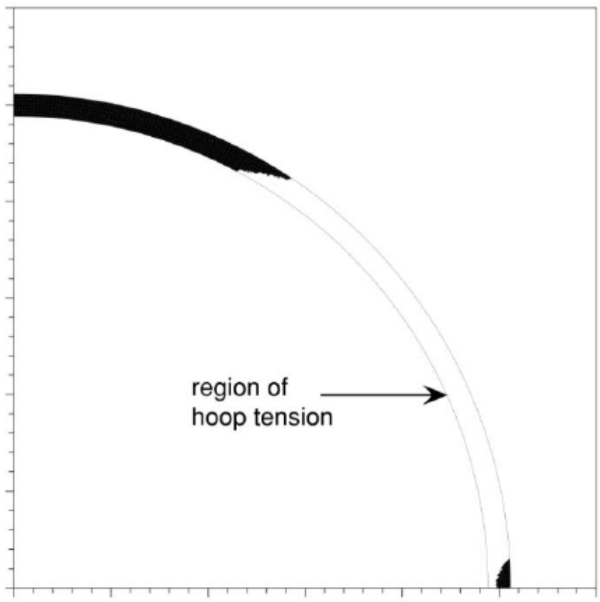

Figure 7: Seventh iteration of analysis, result showing hoop compression and hoop tension region

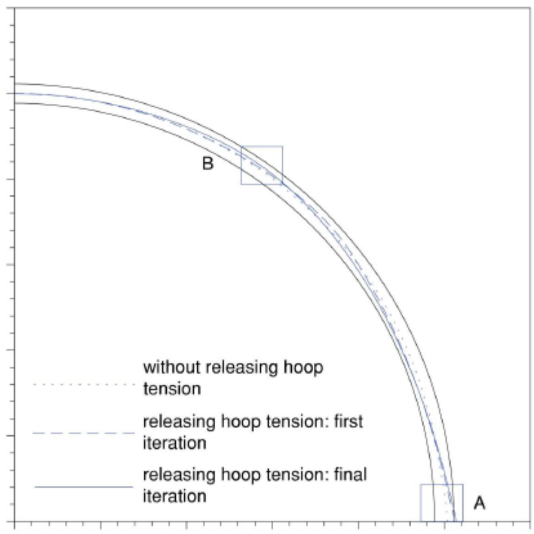

Figure 8: Thrust line for masonry dome with releasing of stiffness corresponding to hoop tension

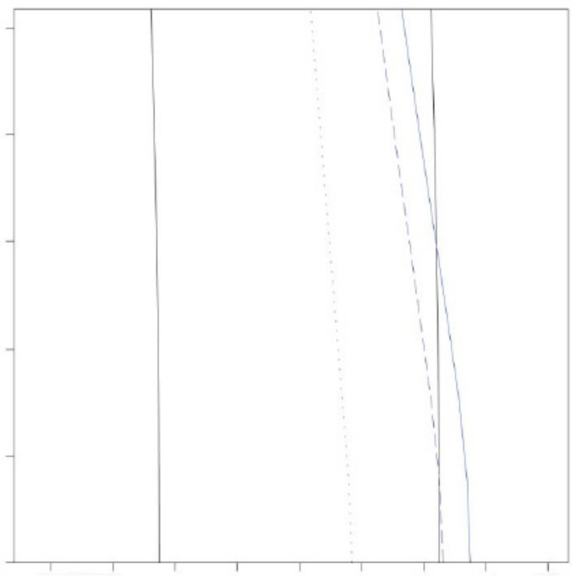

Figure 9: Zoom view at 'A'

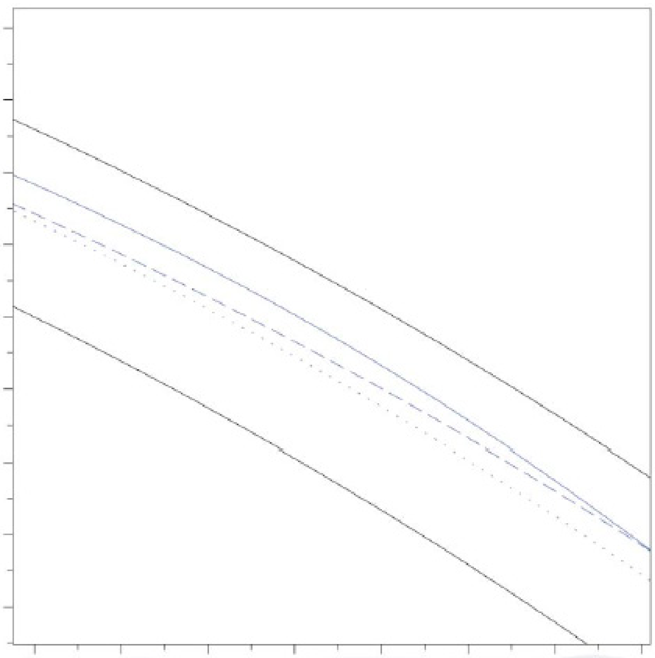

Figure 10: Zoom view at ’B’

CONCLUSIONS

The assessment of the heritage masonry domes for retrofitting purpose requires the appropriate method for the analysis. Specifically in absence of reliable data for mechanical properties of masonry, it poses the greater challenges to the engineers. This paper demonstrate the development of appropriate method of structural analysis, finite element thrust line approach, satisfying following criteria:

1. Masonry structures are critical for stability and not the stress. Hence the stability should be the the primary check as in thrust line method and stress should be the secondary check as considered in finite element analysis. The method presented satisfied these criteria.

2. The material properties of masonry cannot be predicted with great accuracy, and it is apparent from thrust line hypothesis that the stability is almost independent of material properties for practical consideration. This method for stability check is insensitive to mechanical properties of the material and hence it can be used for analysis of heritage masonry dome structures.

3. The method is found versatile to model the intricacy of actual structure.

The proposed method considers the effect of hoop compression forces on the thrust line of masonry dome, and allows the release of hoop tension forces. The presented iterative analysis method gives better approximation for extended hoop tension region and so it gives better understanding of structural stability of masonry dome in presence of the hoop tension cracks.

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Mahesh Varma

Mr. Mahesh Varma is the managing director of Nandadeep Designers and Valuers Pvt. Ltd, Aurangabad. After B. E (Civil Engineering), he pursued hi M. Tech. (Structural Engineering) in IIT Bombay. He has experience of over 15 years in the field of structural engineering. He has remained key analyst behind the success of Global Pagoda.

He is member of syllabus committee (Governing committee) of Government polytechnic, Aurangabad. He is member of scientific committee, domes in the world for the international conference held at Florence Italy. He was member of structural stability committee for the stability study of 'Jalbhavan', the official residence of Honorable Governor of Maharashtra.

He has remained responsible for the many research publications in the field of structural engineering by Nandadeep Designers and Valuers Pvt. Ltd. He has designed PSC (spun concrete) high mast of up to 35 meters for new Mumbai region. He has worked for KEC international and Beton concrete for many Prestress spun concrete (PSC) products for projects in India and abroad. He has also remained associated with RDB, Italy for their precast housing product designs.

PERFORMANCE EVALUATION OF REHABILITATED RC BEAM-COLUMN CONNECTIONS UNDER CYCLIC LOADING

C. Marthong, S. K. Deb and A. Dutta
Department of Civil Engineering, IIT Guwahathi

Abstract: In this paper, the performances of rehabilitated RC beam-column connections under cyclic loading were experimentally investigated. Three full-scale exterior beam-column connections with typical deficiencies namely: (1) beam weak in flexure (2) beam weak in shear and (3) column weak in shear were considered. The joint regions in all the specimens were severely damaged with concrete being totally fragmented during first set of tests. The damaged specimens were then rehabilitated using the removal and replacement technique at the joint region followed by epoxy injection on large cracks. The rehabilitated specimens were subjected to similar cyclic displacement history and their hysteretic responses were obtained. Important parameters related to seismic capacity such as ultimate strength, stiffness degradation, energy dissipation and ductility of the specimens were evaluated. Comparisons of the above parameters with those obtained from the original RC specimens were made. The rehabilitated beam-column connections exhibited equal or marginally better performance and hence the adopted rehabilitation strategy could be considered as satisfactory.

Keywords: Beam-Column Connections; Rehabilitation; Micro Concrete; Epoxy Resin

INTRODUCTION

In the past, numerous reinforced concrete frame structures collapsed due to severe earthquake. Post-earthquake investigations into damaged structures generally showed that in many cases, damages of RC frame structure were localized in beam-column joints which might have led to partial or total collapse of the building. Further, it was observed that the exterior joint, which is confined by only two or three framing beams, had suffered more in comparison to the interior ones.

Several techniques for rehabilitation and strengthening of damaged joints have been reported [1]. Each of the techniques possesses its own practical limitation. Depending on the extent of damages, basic guidelines are available for various repairing techniques [2, 3]. In case of moderately damaged specimens, the epoxy pressure injection technique was recommended. This technique includes injecting of a high strength epoxy under high pressure into the cracked zone of a structural element for the purpose of filling the cracks and adhering to the substrate materials. This ensures the restoration of the bonding of reinforcing bars and the surrounding concrete. In the cases of highly damaged specimens, partial or complete removal of fragmented concrete followed by replacement of voids in damaged zone by appropriate material is required. There are various products commercially available for repair of damaged concrete. Polymer-modified concrete (micro concrete) is the most cost effective one because of its high early compressive, tensile and flexural strength.

This paper evaluates the performance of three classes of damaged deficient RC external beam-column connections, rehabilitated with a simple repair strategy. The scope of this experimental study is limited to testing of three full-scale exterior beam-column connections with three typical deficiencies, namely, (1) beam weak in flexure (BWF) (2) beam weak in shear (BWS) and (3) column weak in shear (CWS), as control specimens. Details of test specimens are shown in Fig.1.

Subsequently, all three damaged specimens were rehabilitated and tested again subjected to similar cyclic displacement history. Results shows that all the rehabilitated specimens exhibited equal or marginally better performance as compared to the corresponding control specimens.

TEST SET-UP AND LOADING PROCEDURE

Test set-up as shown in Figure 2 was used for the testing, where column was placed in horizontal position and beam in vertical position. Both ends of the column were placed on fabricated roller supports in order to simulate the appropriate boundary condition. Constant axial load of 10% of gross capacity of column was applied to the column end abutting against an '*A'* frame. The lateral load was applied on the beam tip by a displacement-control loading system through a 250 kN capacity dynamic actuator having a maximum displacement range of ± 125mm. The displacement amplitudes were gradually increased with three number of loading cycle being repeated at every amplitude of a displacement in both push and pull directions. A typical displacement history is shown in Figure 3.

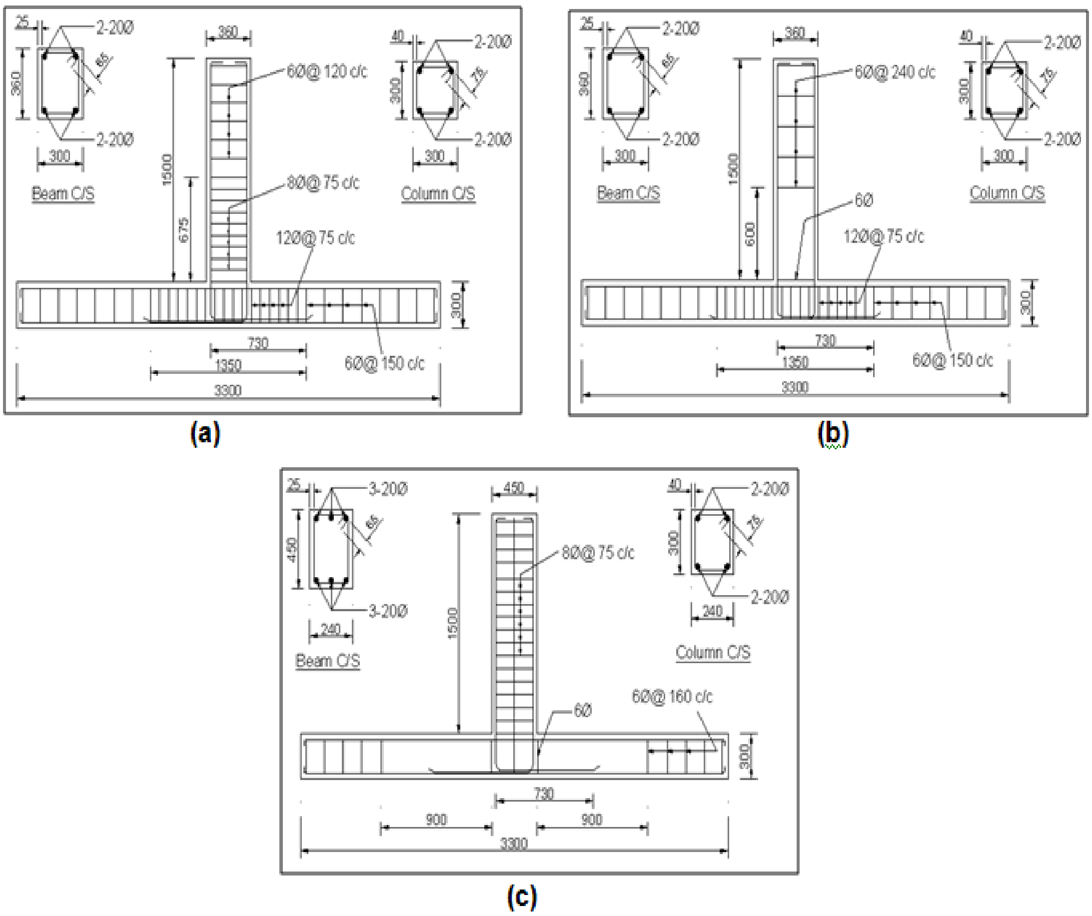

Figure 1: Reinforcement detailing of (a) BWF (b) BWS and (c) CWS

REHABILITATION MATERIALS

A low viscous epoxy resin was used for crack injection. Micro concrete was used as a replacement material. Bonding agent was used for bonding old and newly added concrete. All these materials were procured from Fosroc Chemicals (India) Pvt. Ltd. Further, an injection pump (hand operated) and injection packer manufactured by WEBAC Chemie GmbH were used for injecting epoxy into the cracked zone.

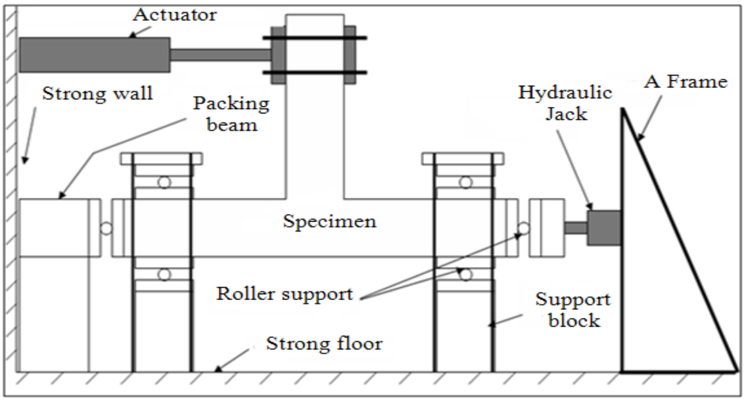

Figure 2: Schematic diagram of test set up

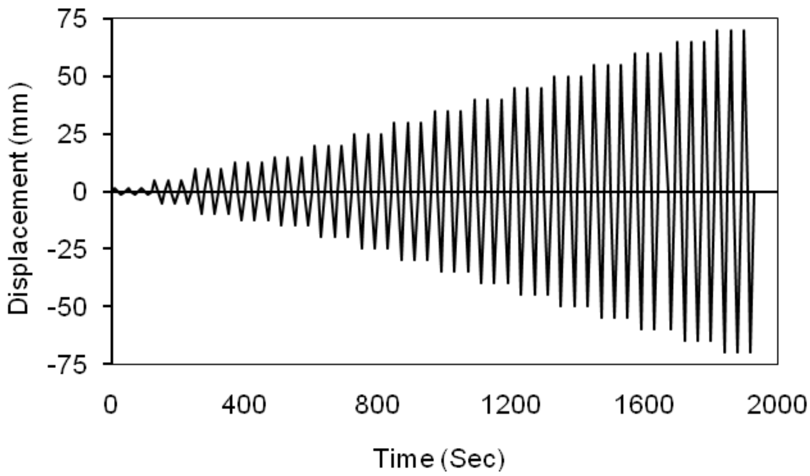

Figure 3: Imposed displacement history

REHABILITATION STRATEGY

Rehabilitation was carried out on the damaged control specimens. First, loose or fragmented concrete from the damaged area were removed and the surface was cleaned using wire brush followed by high-pressure air until the area was free from dust particles. Nitobond EP was applied on the cleaned surface for attaining the adequate bond between old and newly added concrete. The voids in the cleaned area were filled with micro concrete. Holes were drilled along cracks which were left on the surface and packers which serve as filler neck for epoxy injection were inserted through these holes. Visible cracks were sealed by Nitocote VF and a low viscous epoxy resin was injected under high pressure into the cracked zone.

HYSTERETIC RESPONSE

Hysteretic responses of all beam-column connections are shown in Figure 4. Important parameter related to seismic capacity such as ultimate strength, stiffness degradation, energy dissipation and ductility were evaluated from these responses. Comparisons of the above parameters with those obtained from the control specimens were made. Table 1 shows that slightly load carrying capacity, energy dissipation capacity and displacement ductility values were attained by the rehabilitated specimens.

CONCLUSIONS

The performance of the rehabilitated specimens was evaluated by comparing their results with those obtained from the respective control specimens. The comparison reveals that the rehabilitated specimens exhibited slightly higher load carrying capacity, comparable displacement ductility level and similar energy dissipation capacity. Overall, rehabilitated specimens exhibited equal or marginally better performance and hence the adopted rehabilitation strategy could be considered as satisfactory.

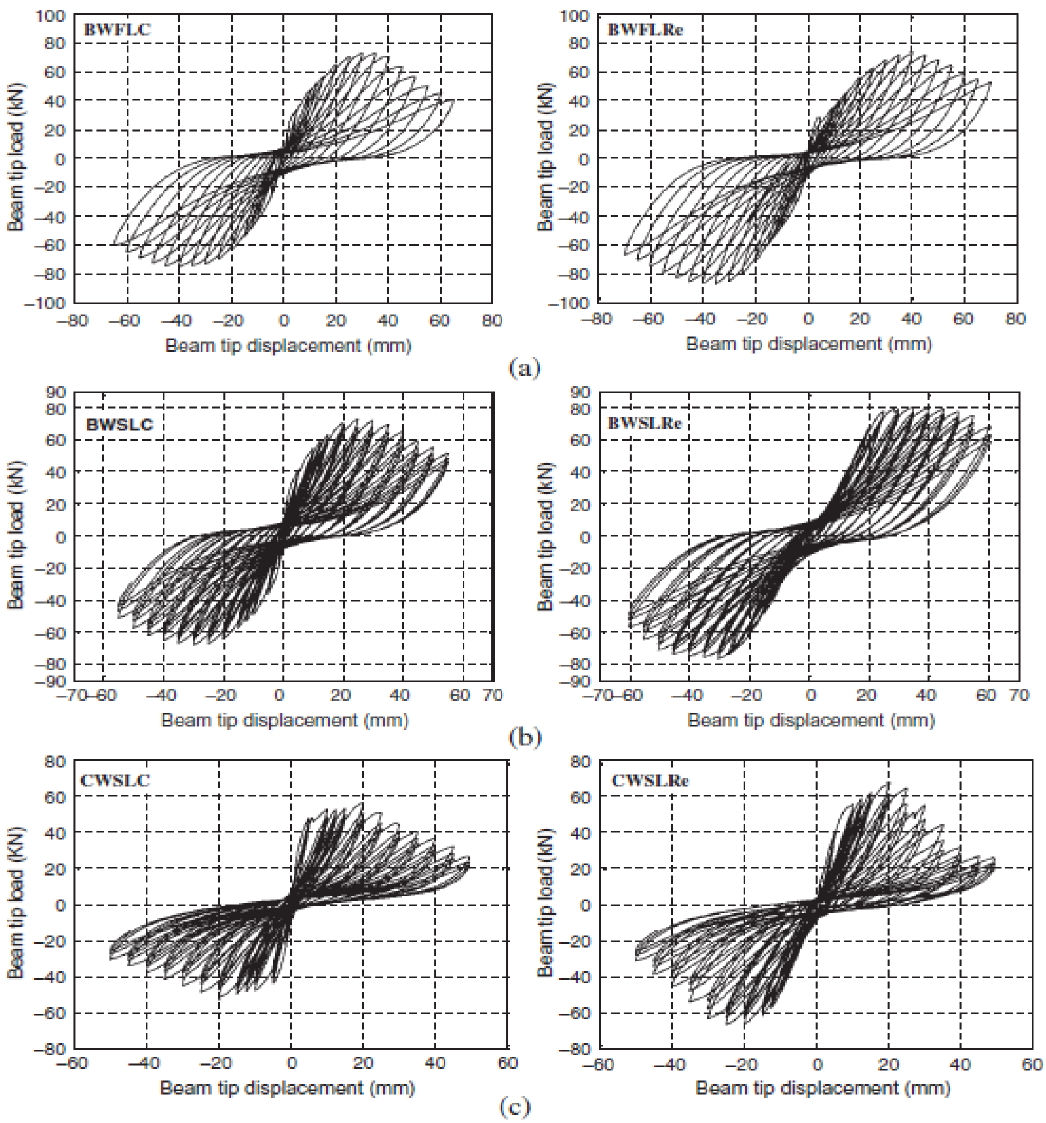

Figure 4: Hysteretic responses of (a) BWF (b) BWS and (c) CWS

Table 1: Comparison of seismic capacity

Specimen	Control			Rehabilitated		
	Load capacity (kN)	Energy dissipation (kN-m)	Ductility (Δ_u/Δ_y)	Load capacity (kN)	Energy dissipation (kN-m)	Ductility (Δ_u/Δ_y)
BWF	73.28	47.18	7.03	84.08	53.91	7.27
BWS	72.08	30.09	8.18	77.22	29.60	9.73
CWS	58.01	13.68	6.20	64.64	14.11	6.80

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Sajal Kanti Deb

Dr. Sajal Kanti Deb currently working as Professor in the Department of Civil Engg., IIT Guwahati. He received his *Ph.D.* in Earthquake Engg. from IIT Roorkee (formerly, UOR) in 1994 and received the award of *Dr. Jai Krishna Gold Medal* during the 17th Engineering Congress organized by Institution of Engineers (India) during December, 2002. He dynamically led the Dept. of Civil Engg., IIT Guwahati during *April 2009 to March 2012 under his headship. He* Planned, designed and developed state-of-the-art dynamic structural testing facility at IIT Guwahati and coordinated 09 nos. of sponsored R&D projects & *FIST grant* from DST during 2011. Ha has published 22 journal publications and 34 conference publications and 1 book chapter. He has guided 4 PhD students and 6 students are currently pursuing PhD under his guidance. He was the Chairman - GATE 2003 Examination (IIT Guwahati). His research area includes Passive and semi-active control, Performance based seismic design, System identification & structural health monitoring, Seismic Retrofitting. He is a member of International Panel /National Committees such as Editorial Advisory Panel, *Structures and Buildings*, Proceedings of Institution of Civil Engineers (ICE), UK, Editorial Board, ISET *Journal,* Indian Society of Earthquake Technology, Member of Working Committee of Experts on *Vulnerability Analysis and Risk Assessment*, National Disaster Management Authority (NDMA), GOI, Member – Assam Govt. Hazard safety Cell.

STRUCTURAL RETROFITTING : CASE STUDY

A. K. Singh

Abstract: This paper describes the intensive retrofitting of an RCC framed structure under construction. The building is a multi-storeyed Triple Basement + Ground + 10 upper floors RCC framed building meant to be used as an institute building. The building was taken up for retrofitting purpose owing to the design and detailing error. The paper emphasizes the various stages like planning, diagnosis, methodology and execution of retrofitting work.

Keywords: PT slab, Tendons, USPV, Load test, Carbon laminate, Pressure grout, carbon laminate,

INTRODUCTION

Recently at one construction site, during routine post form work stripping check up of Post Tensioned slab a noticeable crack was observed in the pre-stressed (Post-tensioned) slab top. At the time of inspection, 1st to 5th floor level slabs were already cast and preparation for casting of 6th floor level slab was in progress. Being a public building meant for a prominent institute, the construction work was suspended till the problem was diagnosed and corrections were made in design and details. Following action was taken immediately.

- Prop the slab from below.
- Place the thin glass plates at the crack portion to monitor the movement of crack.
- Conduct USPV test.
- Inspect the other locations as well for the crack appearance.

Having inspected at other locations, identical cracks were observed.

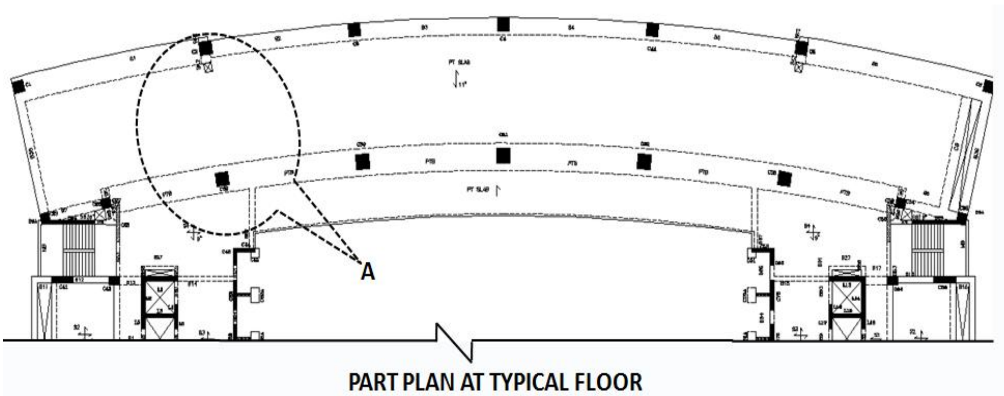

Fig. 1: Part Plan at Typical Floor

INVESTIGATION: Ultrasonic Pulse Velocity tests:

Ultrasonic Pulse Velocity tests were carried out to qualitatively assess the homogeneity, integrity of concrete and depth of cracks. The scheme of test was to measure the USPV along cracks, parallel to cracks and at sound locations. A total of 32 locations were tested. The concrete near the cracks (parallel and across) were showing either no reading or very low USPV. This was indicative of the presence of cracks not only in the visible cracked region but also in the region where apparently there were no visible cracks. Passage of waves despite of cracking was due to presence of reinforcement. At sound locations also, the USPV was less than 3.0 km/s. Having related the same with the site cube strength, the underlying reason for low USPV was the presence of trapped moisture and usage of more than 42% fine aggregates. The RCC consultant was of opinion that the cracks have appeared because of low integrity concrete as obtained from USPV.

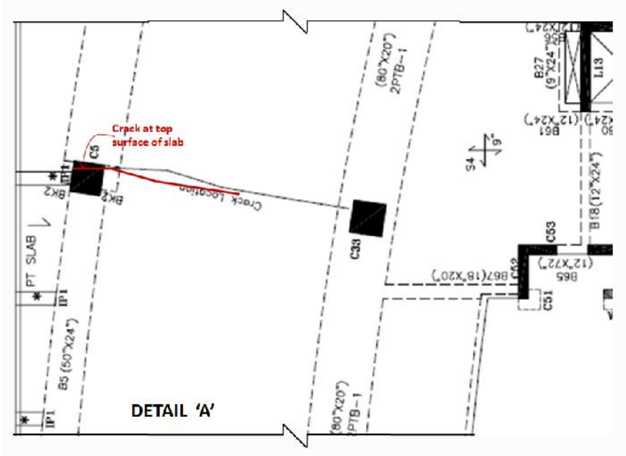

Fig. 2: Detail at 'A'

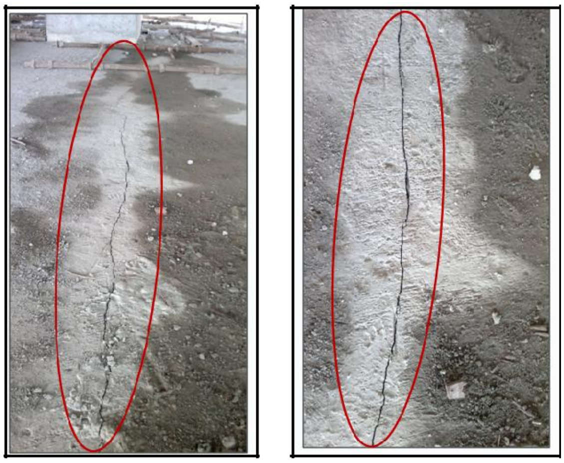

Fig. 3: Full depth crack in P.T.Slabs

Investigation by record:

The records from site for this location were retrieved. The record found at site for this particular concrete pour on North-West corner at 2nd floor level are as under.

- Location = 2nd floor
- Date of casting = 02-09-2011
- Date of stressing = 22-09-2011 at an age of 21 days
- Concrete grade in slab = M40
- Cube strength at the time of stressing = Adequate (< 25 MPa)
- Average 28 days cube strength = 44.88 MPa
- Deviation between individual results < 8%
- Steel used = TMT Fe 500

The investigation by record showed that there were no issues pertaining to the workmanship. The slab was casted as per the details and work specifications. Cube compressive strength results, steel test results, slab checking, pour card, etc. were in order. Hence possibility of any lapse on workmanship was ruled out.

Further investigation:

The flat slab was having

- Grid spacing Radial, 12.5m x 9.7m
- Depth of slab 275 mm with PT / RCC beam
- Column size as per detailed drawing

The checking was limited to the flat slab design, of typical floors, where cracks were observed. The following documents were provided for reviewing:

- Set of architectural drawings.
- Set of structural drawings
- Structural Design Basis Report (DBR) and analysis file of typical floor slab.

Since at few locations the Cracks were observed, it was agreed to review the design under the existing load condition and predominantly for gravity loads. As the cracks were observed before the application of the live loads, the lateral load effect was not counted in study. For the analysis model creation and loading, following parameters in accordance with the structural consultant's design philosophy, were considered.

Construction Material:

a. Concrete complies with IS 456:2000. The grade of concrete used for the Slabs and Beams = 40 N/mm^2

b. Non-prestressed steel reinforcement: Yield Strength : 500 N/mm^2 and Modulus of Elasticity = 200,000 N/mm^2

c. Post-tensioning Steel Strands: (1). Material : As per IS 14268-1995, (2). Low relaxation, seven wire strand, (3). Strand relaxation after 1000 hrs : 2.5%, (4). Strand diameter : 12.7 mm, (5). Strand Area : 98.7 mm2, (6). Modulus of Elasticity (E_s) : 1,95,000 N/mm^2, (7). Ultimate strength (f_{pu}) : 1,860 N/mm^2, and (9). Min. Breaking strength : 183.7 KN

Loading:

Following load cases with the detailed description of each of the load cases were considered:

Dead Load: The dead load comprising of self weight of the structure and loading due to finishes and floorings etc. which are permanent in nature as per IS 875 Part (I):1987

1. Finishes including services load = 1.50 KN/m^2
2. Density of Concrete = 25 KN/m^3
3. Density of Brick = 22 KN/m^3

Live Load: Following Live loads were considered on floor slabs as per IS 875 Part (II):1987.

1. General Floor area of Institute = 4.0 KN/m^2
2. Corridors, Lift Lobbies = 4.0 KN/m^2
3. Wash Room (Toilet areas) = 2.0 KN/m^2
4. Staircases = 4.0 KN/m^2

Load Combinations

Primary Loads and Load Combinations

1. Primary Load cases:

a. Dead Load (D. L.)

b. Live Load (L. L.)

2. Basic Load combinations:

a. 1.5(Self weight)

b. 1.5(Self weight + F.F. + L. L)

Pre - Stressing Criteria

1. Pre-stressing system :

a. System : Bonded (Grouted)

b. No of strands : 5 per duct

2. Pre-stressing losses: Friction calculated in accordance with IS:1343

a. $P_x = P_o . e^{-\mu(\alpha+\omega x)}$

b. P_x : Force at distance x from stressing end

c. P_o : Stressing force

d. μ : Angluar friction co-efficient = 0.2 (Bonded system)

e. α : Angle change in tendon from anchor to point considered (radians)

f. Wobble factor = 0.0046 radians/m

3. Stressing of Tendons :

Final Jacking force = 137.8 KN

Methodology:

For analysis and design CEDRUS – finite element software, capable of designing the floor slab plate for PT effect, was used. The Loads were applied as indicated above. Finite elements are created automatically by software. PT cable layout was prepared as per the 'Valid for construction drawings', where cables followed elastic BM profile. It is considered that cables once stressed, started taking gravity loads from slab inducing the secondary bending moments. Depending on the PT cable force and elastic BM, the additional reinforcement required at particular section was calculated for Strength analysis and serviceability. Serviceability design has been performed by limiting the rebar stress to 280 MPa.

Check for Deflection:

Max. Short term deflection from output: 24 mm Long term deflection : 3.0 (creep factor) x 24 = 72mm >Allowable.
Max. Allowable: Span / 250 = 15600 / 250 = 62 mm

The dimension of building was 83m x 48m. There is no expansion joint proposed. To eliminate the possibility due to temperature load, the analysis was carried out for temperature load of 15^{0}C and it was found that the additional moment induced was less than 3KN.m, which was not significant to cause any cracking. Hence the possibility of cracking due to no expansion joint was ruled out. SAFE 12.2.0 pro-gram was used to analyze and design the typical floor flat slab. The bending moment contours are given in figures (4) to (11) are showing the contour for solid slab between node points C-5, C-33, C-32 and C-4A.

1. Fig (4) & (5) shows (+)ve moment along x-axis.
2. Fig. (6) & (7) shows (+)ve moment along y-axis.
3. Fig. (8) & (9) shows (-)ve moment along y-axis.
4. Fig. (10) & (11) shows (-)ve moment along x-axis.

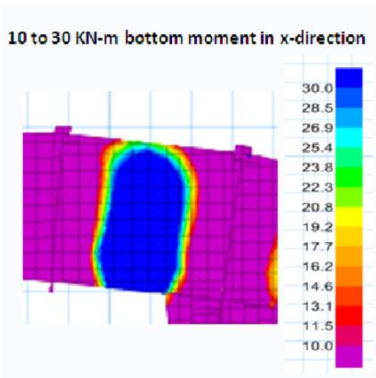

Fig. 4

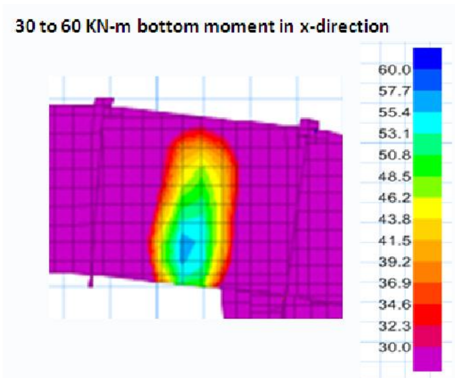

Fig. 5

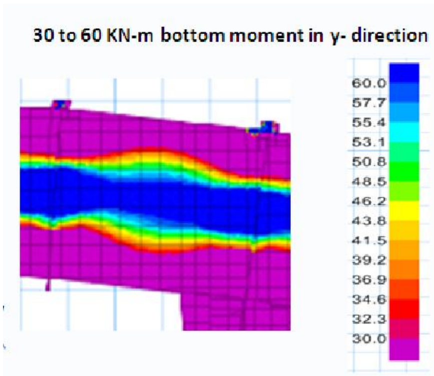

Fig. 6

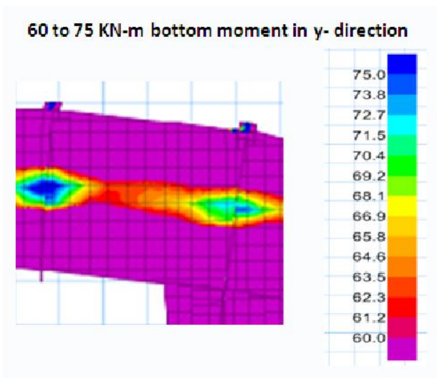

Fig. 7

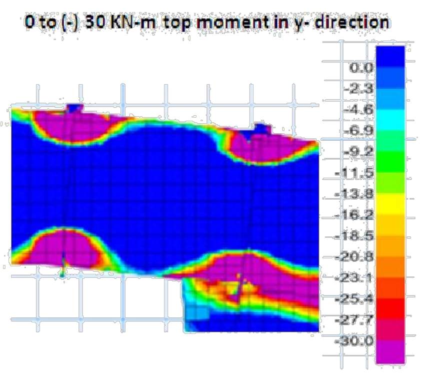

Fig. 8

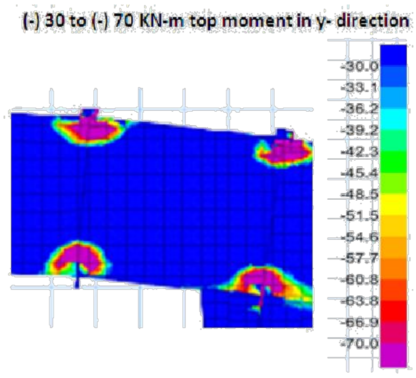

Fig. 9

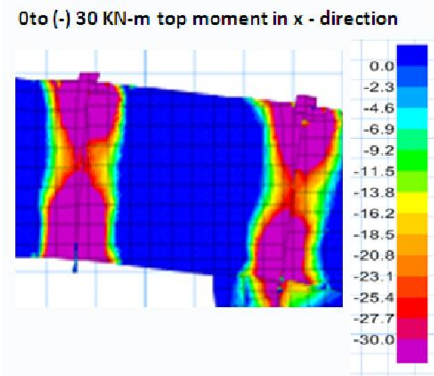

Fig. 10

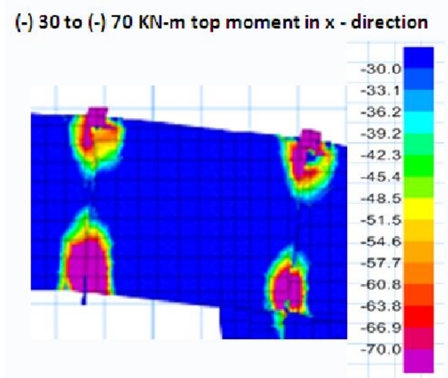

Fig.11

From the available 'Valid for construction' drawings, the unfactored moment of resistance of the slab section with available nominal reinforcement of # 8mm @ 150mm c/c along x-direction at centre (mid span) in sagging is 23.03 KN.m, while at column strip along y-axis, the unfactored moment of resistance of the slab sec-tion along x-direction in hogging 0 KN.m as there was no reinforcement available at top in this strip.

Findings:

Figures (4) to (11) gave a conclusive indication that:

a) Long term deflection is slightly more than the allowable, which can be reduced by providing adequate camber at site.

b) The hypothesis that slabs, though having value of β (L_y / L_x ratio) less than 2, could be designed as one-way. The basic behavior of plate would be two-way action till it failed to find the desired stiffness and cracks. Once cracked the behavior of slab would be one-way and given PT design stood true.

c) The bottom reinforcement nominally provided at # 8mm @ 150mm c/c along x-direction at centre (mid span) was adequate for the moment in the range of 10 KN.m to 30 KN.m. In y-direction, the moment range of 30 KN.m to 60 KN.m (Refer Fig-6) in central strip and 62 KN.m to 68 KN.m, for which the section was inadequate. This was the point of attention.

d) Top reinforcement was not available in the x-direction. Between columns C-5 & C-33, the moment contours indicate that majority of values were in the range of 30KN.m (Refer Fig-10). The hypothesis of one way behavior did not war-rant any reinforcement but factually the negative reinforcement at top in x-direction was required, which was missing. Near columns, the moment value was in the range of 40KN.m to 70KN.m (Ref. Fig-11). This led to cracking in slab between columns C-5 & C-33.

e) Investigating the Fig.-8, 9, 10 and 11, there was a very clear indication that a column capital formation was taking place due to rigidity of columns. For this formation, there was no provision in drawing details. This was another point of attention.

Based on findings of analysis it was decided to investigate the slabs further, before adopting any retrofitting scheme.

Experimental investigation (Load Test):

Scheme: Scheme of load test was prepared on following counts:

a) Load test on slabs which had not exhibited any distress till date but analysis was showing them doubtful accounting the fact that load test may distress the apparently sound slab.

b) Load test on slabs which had not exhibited any distress till date before retrofit-ting based on analysis results

c) Load test on distressed slabs after retrofitting the same to understand the effectiveness of retrofitting measures.

The load test was conducted over the 3rd floor level slabs between column grids C91/106, C92/107, C93/109 and C94/110 in order to ascertain the structural safety under the anticipated loads as per IS 456-2000 and IS 875-Part 1 & 2.

Instruments:

Instruments used were Linear potentiometer for Deflection Measurement (Conductive Plastic Linear Potentiometers, Conductive Plastic Linear Potentiometers and Mechanical Lever Dial Gauge), Readout unit (Microprocessor Potentiometer, Level, Position & Set-ting Digital Meter), Instrument for measuring behavioural parameters and Omega type Transducer for Crack width measurement.

Load Arrangements:

(a). Loading: In order to ensure the load distribution over the slab uniformly, the structure was loaded using water up to required height.

(b). Placing of dial Gauges:

The Dial Gauges were fixed at mid span of each bay of slab and beam in order to measure the deflections. The locations of dial gauges are shown in Fig. (11).

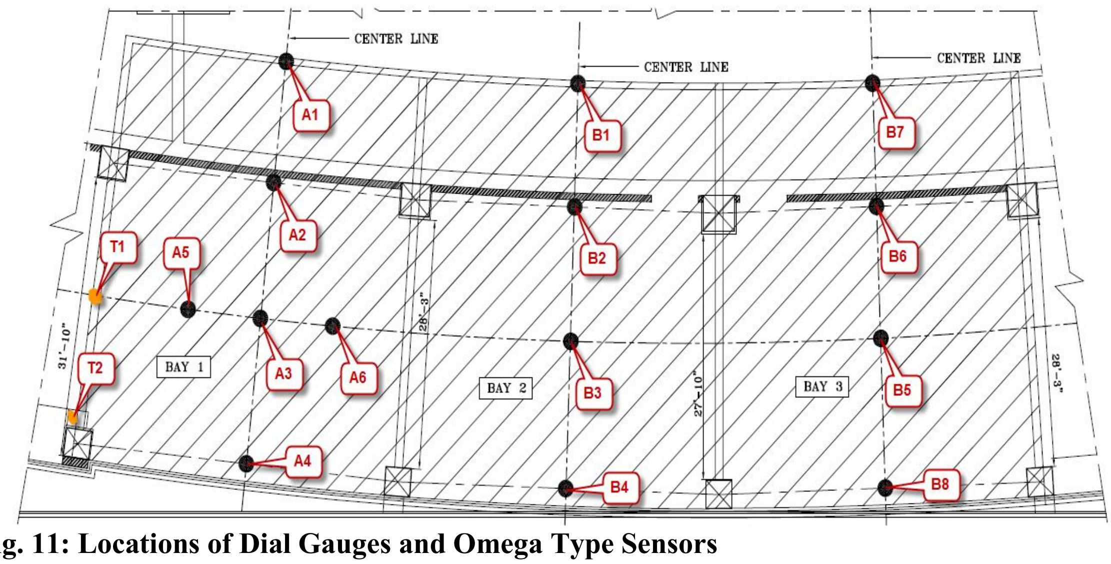

Fig. 11: Locations of Dial Gauges and Omega Type Sensors

Test Load Calculations:

As per the recommendations, the test was carried out for the total expected dead load + 1.25 times imposed loads.

Load Calculation based on Design Calculation is as below: Total Load = Dead Load + (Live Load x Factor)

= $4\ KN/m^2 + (3\ KN/m^2 \times 1.25)$

= $407.75\ Kg/m^2 + (305.8\ Kg/m^2 \times 1.25)$

= 790 Kg / $m^2 \approx$ 8.00 KN/m^2

Sequence of testing:

The test was performed simultaneously on 3 spans of the slab. The load was applied on all spans simultaneously. Following 3 load combinations were applied to slab.

Case 1: Design dead load on all spans with full design imposed load on all spans (Fig.12a), Case 2: Design dead load on all spans with full design imposed load on 1st span (Fig.12b), and

Case 3: Design dead load on all spans with full design imposed load on 1st and 3rd span. (Fig.12c)

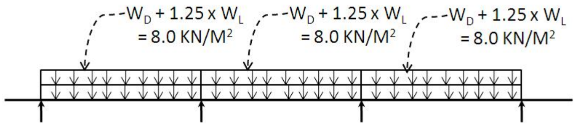

Fig.12a: Load Case-1

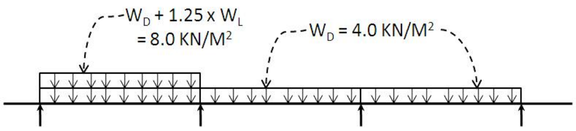

Fig.12b: Load Case-2

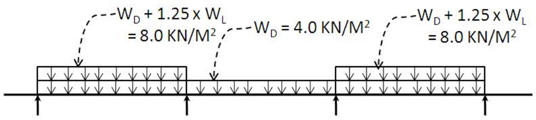

Fig.12c: Load Case-3

Sensors were mounted (omega type crack width transducers) for monitoring (measuring) increase in crack width at top face of the slab at the location (T1, T2) as shown in Figure 11. Results from the reading of these sensors as well as visual observations indicate that the cracks were widened at higher loads.

Performance Criterion:

1. The deflection due to imposed load only shall be recorded. If within 24 hours of removal of the imposed load the structure does not recover at least 75 percent of the def-lection under superimposed load, the test may be repeated after a lapse of 72 hours. If the recovery is less than 80 percent, the structure shall be deemed to be unacceptable.
2. If the maximum deflection in mm, shown during 24 hours under load is less than $40L^2/D$, where 'L' is the effective span in m; and D, the overall depth of the section in mm, it is not necessary for the recovery to be measured and the recovery provisions of point (1) above shall not apply.
3. The surface width of the cracks should not, in general, exceed 0.300 mm in members where cracking is not harmful and does not have any serious adverse effects upon the preservation of reinforcing steel nor upon the durability of the structures. In members where cracking in the tensile zone is harmful either because they are exposed to the effects of the weather or continuously exposed to moisture or in contact soil or ground water, an upper limit of 0.200 mm is suggested for the maximum width of cracks. For particularly aggressive environment, such as the 'severe' category in Table (3) of IS-456-2000, the assessed surface width of cracks should not in general, exceed 0.100 mm.

Test results

(A): Deflection:

Loca-tion	*Type*	*L, m*	*D, mm*	*$40L^2/D$, mm*	*Load Case 1*	*Load case 2*	*Load case 3*	*Re-marks*
A3	PT Slab	9.79	275	13.94	3.469	4.418	4.343	Passes
B3	PT Slab	9.79	275	13.94	3.247	1.3255	0.616	Passes
B5	PT Slab	9.79	275	13.94	3.195	4.016	0.572	Passes
A2	PTB	11.75	500	11.05	2.105	2.3475	2.085	Passes
B2	PTB	11.75	500	11.05	1.953	1.705	0.459	Passes
B6	PTB	11.75	500	11.05	1.885	2.3065	0.467	Passes
A4	RCB	12.60	600	10.58	0.801	1.262	1.000	Passes
B4	RCB	12.60	600	10.58	0.765	0.407	0.154	Passes
B8	RCB	12.60	600	10.58	0.822	1.177	0.147	Passes
A1	PT Slab	6.80	275	06.73	1.581	2.013	1.420	Passes
B1	PT Slab	6.80	275	06.73	3.230	1.776	0.865	Passes
B7	PT Slab	6.80	275	06.73	2.830	3.084	0.573	Passes

Note: In case of cantilever slab, the span is taken as 2 x the length 3.40m

Inference: Reference to performance criterion (2) the maximum deflection in mm, shown during 24 hours under all 3 cases of loading were less than $40L^2/D$, where 'L' is the effective span in m; and D, the overall depth of the section in mm, hence it was not necessary for the recovery to be measured and the recovery provisions of point (1) above was not applicable and slabs / beams were qualifying the performance criteria in deflection.

(B): Crack width:

Location	Load Case 1 Crack width Opening	Load Case 2 Crack width Opening	Load Case 3 Crack width Opening
T1	0. 328 mm	0.641 mm	0.558 mm
T2	1.227 mm	1.798 mm	1.521 mm

Inference: Reference to performance criterion (3) the surface widths of the cracks were in excess of permitted values and slab band between the columns did not qualify the performance criteria in crack width. The performance criteria limits the crack width 0.300 mm in members where cracking is not harmful and does not have any serious adverse effects upon the preservation of reinforcing steel nor upon the durability of the structures. The structure was situated in aggressive environment, under severe category; the assessed surface width of cracks should not exceed 0.100 mm. Severe category of environmental expo-sure reads, "Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation Concrete completely immersed in sea water, Concrete exposed to coastal environment." Further these cracks were live cracks, which varied by 0.303mm and 0.517mm at respectively locations T1 and T2.

REMEDIAL MEASURES:

Based on Investigation by record, re- analysis and load test, it was confirmed that the cracks in the slabs were developed due to lack of detailing at negative moment location of the slabs. The analysis of the slabs have indicated that there were significant negative moments developed even after tensioning in one direction and there was no negative reinforcement to resist it which resulted in the cracks after casting and removal of shuttering.

The detailing error was to be supplemented by externally bonded reinforcement either steel plates or carbon laminates. Steel plates were prone to corrode under environmental condition, it was recommended to opt for externally bonded carbon laminates in combination of non-prestressed and prestressed carbon laminates as per schematic diagram is shown in Fig. (13) and Fig. (14) . It was recommended that after laminate application, high pressure grouting with low and very low viscosity epoxy / high molecular weight monomer should be done to fill the cavity / cracks.

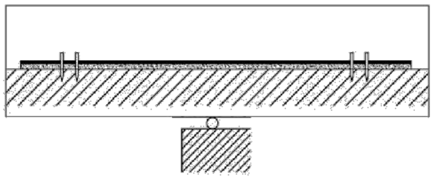

Fig.13: Non-prestressedcarbon laminate

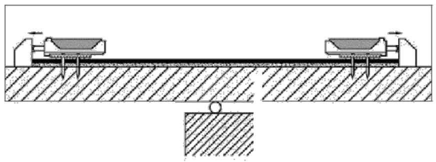

Fig.14: Pre-stressed carbon laminate

The slabs of 6th floor and above of the building shall be casted with the following modified detailing of the steel reinforcement, where additional reinforcement shall be provided to resist the negative BM in column strip.

Design of Retrofitting scheme:

1. Design Loads

$M_u^{(-)}$ = 45 kNm

$M_u^{(+)}$ = 0 kNm V_u = 0 kN

2. Material properties

Grade of Concrete f_{ck} = 40 N/mm2 Grade of Steel f_y = 500 N/mm2

f_f = 3250 N/mm2

3. Section Properties

@ **Support:**

Total depth of beam D = 275 mm Effective depth of beam d = 246 mm Breadth of beam b = 1000 mm

d' = 0 mm

Area of Steel in tension A_{st} = 0 mm^2 (As No Reinforcement is provided at top) Area of Steel in Compression A_{sc} = 0 mm^2

4. Evaluation of capacity:

(a). Bending moment capacity at support

Neutral Axis depth (from trial and error), x_u = 0.1 mm

Strain at the level of compressive reinforcement, $\varepsilon_{sc} = 0.0035*(x_u - d')/x_u$ thus, ε_{sc} = 0.0035 mm/mm,

Corresponding stress in the compression reinforcement, f_{sc} = 353.00 N/mm^2 Compressive force in concrete, $C_c = 0.36 f_{ck} b x_u$

thus, C_c = 1.44 kN

$C_s = f_{sc}A_{sc}$ = 0.00 kN

Tensile force in tension steel, $T_s = 0.87 f_y A_{st}$ = 0.00 kN Total Compressive force, Cc+Cs = 1.44 kN

Since, total compressive force, (Cc + Cs) ≈ total tensile force (Ts), the neutral axis depth is correct.

Moment of resistance of the section, $Mu^{(+)}$,all = $Cc(d - 0.42x_u) + C_s(d - d')$ thus, $Mu^{(+)}$,all = 0.35 kNm< Required 45 kNm,

…………… Hence, the section is Unsafe and Strengthening is required.

(b). Bending moment capacity at mid span

Neutral Axis depth (from trial and error), x_u = 0.1 mm

Strain at the level of compressive reinforcement, $\varepsilon_{sc} = 0.0035*(x_u - d')/x_u$ thus, ε_{sc} = -1.7465 mm/mm, Corresponding stress in the compression reinforcement, f_{sc} = 353.00 N/mm^2 Compressive force in concrete, $C_c = 0.36f_{ck}bx_u$

thus, C_c = 1.44 kN $C_s = f_{sc}A_{sc}$ = 0.00 kN.

Tensile force in tension steel, $T_s = 0.87f_yA_{st}$ = 0.00 kN Total Compressive force, Cc+Cs = 1.44 kN

Since, total compressive force, (Cc + Cs) ≈ total tensile force (Ts), the neutral axis depth is correct.

Moment of resistance of the section, $Mu^{(+)}$,all = $Cc(d – 0.42x_u) + C_s(d – d')$ thus, $Mu^{(+)}$,all = 0.35 kNm< Required,

…………… Hence the section is safe and Strengthening is not required.

5. Design of strengthening @ support:

The design moment $M_u^{(-)}$ = 45 kNm

Design stress for carbon laminates(with anchors), f_f = 1500 N/mm^2 Let the area of the laminates required be A_f

The neutral axis depth for the strengthened section is given by (assuming that the compression reinforcement has yielded)

$x_{u,str} = (0.87f_yA_{st} – f_{sc}A_{sc} + f_fA_f)/(0.36f_{ck}b)$

The moment carrying capacity for the strengthened section is given by,

$Mu,str = 0.87fyA_{st}(d – 0.42x_{u,str}) + 0.87*f_y*A_{sc}*(0.42*x_{u,str}–d') + f_fA_f(D – 0.42x_{u,str})$

Now, for $M_{u,str} = Mu^{(+)}$ = 56.46 kNm By simplifying and solving for A_f we get,

Area of the laminates required, A_f = 140 mm^2

Using laminate of 100/1.4, we get, number of laminate required as = 1 Therefore, provide 1 laminate of 100/1.4 per metreupto 3.0m on either side of the support at top i.e provide at 750 c/c.

@ mid span:

The design moment $M_u^{(-)}$ = 0 kNm

Design stress for carbon laminates(with anchors), f_f = 1500 N/mm^2 Let the area of the laminates required be A_f

The neutral axis depth for the strengthened section is given by (assuming that the compression reinforcement has yielded)

$x_{u,str} = (0.87f_yA_{st} – f_{sc}A_{sc} + f_fA_f)/(0.36f_{ck}b)$

The moment carrying capacity for the strengthened section is given by,

$Mu,str = 0.87fyA_{st}(d – 0.42x_{u,str}) + 0.87*f_y*A_{sc}*(0.42*x_{u,str}–d') + f_fA_f(D – 0.42x_{u,str})$

Now, for $M_{u,str} = Mu^{(+)}$ = 0 kNm

By simplifying and solving for A_f we get, Area of the laminates required, A_f = 0 mm^2

Using laminate of 100/1.4, we get, number of laminate required as = 0 Therefore, no laminate is required at mid span at bottom.

TECHNICAL SPECIFICATIONS

(Fig. 15 to Fig.22)

(1) FRP Composite System:

a. Material:

The fibre fabric should essentially be a uni-directional fabric comprising of high strength continuous fibres oriented with E-glass / carbon fibres (as indicated) in the primary direction. The fibre reinforcement fabric should be woven and bounded such that there should be no disturbance of the main and secondary direction fibres upon tai-lor cutting, saturation and handling prior to and during the wrapping operation. The fi-bre reinforcement fabric should be made available in the rolls of minimum 300mm width and a continuous length of minimum 30 meters.

b. Resin:

Appropriate formulated two component epoxy resin compatible to fibre fabric to serve the purposes of surface priming and saturation should be used.

c. FRP Shear Anchors System:

In case aspect ratio is high or the dimension of structural element is large then for effective load transfer and confinement FRP composite system should include an anchoring system. The anchoring system should be of the same composite material. The diameter of the anchor should be between 12 mm. to 15 mm. and should have a minimum embedded length of 75 mm. to 150 mm. from the wrap surface. The FRP anchor should be pre-cured before insertion. The tail of anchor should be well spread over the wrapped surface.

e. Properties of FRP composite:

Evaluation of FRP composite systems shall be based on the specifications and details provided by the manufacturers. The evaluation shall be on the basis of characteristics of fibre fabric, properties and characteristics of epoxy primer and saturant resins and properties and characteristics of cured composite laminate. During the work progress the data sheet of fibre shall be submitted shall be validated by test reports.

f. FRP Composite System Installer Competency Criteria:

The contractor shall provide complete details of the installation process, quality control procedures and other relevant details pertaining to the FRP composite system.

The contractor should have undergone training and he should be certified as trained applicator by the system manufacturer. The training should include surface preparation, material handling, batching and mixing of epoxy resin, manual and mechanical saturation, installation, curing and preparation of specimen for testing.

g. Method Statement:

(i). Non metallic composite fiber wrapping:

• Mortar Treatment: Application of average 10 to 20 mm thick Polymer Modified Mortar over the uneven RCC element surface by using Polymer Modified Cement bonding coat.

• Curing of subsurface: Mortar should be cured properly, maintaining moisture level in substrata less than 4 to 6%.

• Surface Preparation: The surface to be repaired is ground to smooth out the irregularities and sharp corners. Rounding of column / beam edges shall be done by grinding.

• Application of Primer: In order to improve adhesion and prevent the surface from drawing resin from the wrap, a low viscosity epoxy primer is applied with a roller until the substrate is locally saturated.

• Application of Putty: An adhesive, high viscosity putty is applied when necessary to the surface to fill in pin holes, offsets or voids.

• Application of Saturant: Apply Saturant coat to primed surface and FRP sheet using a medium nap roller. (Mechanized wetting of fiber sheet: Cut the fabric to required size. Mix the saturant and fill the same in tub area of the wet lay up saturation ma-chine. Place the fiber on metallic rods and pass through the rollers of machine and saturant tub ensuring complete wetting (65% fiber volume fraction) of fibers. Remove the fiber on rod and place the new fiber sheet.)

• Application of Composite Wraps: Fabric will be carefully laid onto the surface and smoothed out to remove air bubbles and ensure that the fibers are straight and there are no wrinkles.

• Sand Sprinkling: Sand should be sprinkled on the final coat of wrap.

(ii). Non metallic non-pre-stressed pre-cured carbon fiber laminates:

• Mortar Treatment: Application of average 10 to 20 mm thick Polymer Modified Mortar over the uneven RCC element surface by using Polymer Modified Cement bonding coat. Curing of subsurface: Mortar should be cured properly, maintaining moisture level in substrata less than 4 to 6%.

• Surface preparation: Grind concrete substrate, cleaning it with wire brush removing oil, laitance if present, etc. complete.

• Profiling: Apply compatible primer on prepared substrate, Fill the holes and uneven surface with thixotropic putty etc. complete

• Application of plate: Mark the application area on structural element, cut the plate to require size, apply compatible structural adhesive on plate in parabolic manner by adhesive laminating machine (gluing machine), paste the laminate on desire area by using tamping roller to avoid any air voids etc. complete.

- Sand pasting: Apply second coat of saturant after min. 12 hrs, rectify air voids if any paste the river sand on it to make surface rough to take any further finishes.

(iii). Non metallic pre-stressed pre-cured carbon fiber laminates:

It's a new technical system for strengthening the slab and beams. The pre-stressing ma-chine has a tonnage capacity to stretch the laminate from 0 to 24 tons whereas applied load will be 6 to 8 tons at this site. All procedure will be same as (ii) above except pre-stressing process.

- Drilling & Fitting work of Pre-stress machine parts on Slab / beams shall be done be-fore applying the Load.
- All the fixtures, anchor bolts plates, etc. shall be thoroughly checked before applying the pre-stressing force.
- Apply the pre-stressing force by hand operated hydraulic pre-stressing machine.

(2) Stitching of cracks:

When RC members have honey combing inside after removal of loose concrete material it is advised to inject low viscosity epoxy injection resin / high molecular weight monomer in the honey combed area. Also same technique could be adopted in case of cracks (2 to 3 mm. wide) in these elements. Drilling holes in concrete and fixing injection nozzle using with ready to use polymer based quick stopper plug (sealer) . A hole shall be drilled into the honey combed area and along the cracks. Compressed air shall be used to clean the hole, cracks and honey combed portion to remove dust and dirt. One way injection packers shall be fixed into the holes. Spacing of holes shall be 12″ to 18″ depending upon the deficiency / crack pattern. Surface of crack shall be sealed with epoxy sealant to ensure that injection material does not leak from cracks. Also the ho-ney combed areas shall be sealed in the similar manner. Injection packers shall be se-cured in fixed position with help of epoxy sealant. This preparation shall be left for 24 hours for drying and hardening.

Injection of epoxy resin shall be completed in two stages using Very Low Viscosity Injection Resin and Low Viscosity Resin / high molecular weight monomer with 24 hrs operation interval. Very Low Viscosity Injection Resin/ high molecular weight monomer shall be injected to fill very fine cracks and micro-cracks and also to strengthen the injected portion. Low Viscosity Resin shall be injected to finally fill the micro-cracks and hone combs. Both operations shall be used in combination. Injection pressure shall be 2 kg/cm2 for Very Low Viscosity Resin and 2.5 to 3.0 kg/cm2 for Low Viscosity Resin / high molecular weight monomer. Hand operated compressor shall be used with small air flow. In case no material could be injected from one Injection packers due to path blocked, maintain the pressure for 10 min. and continue the operation from next Injection packers. Having completed the injection process the Injection packers shall be flush cut after 24 hours."

PHOTO EXHIBITS


Fig. 15, 16: Water ponding for uniform load

Fig. 17: Pedestals for dial gauges

Fig. 18: Linear Potentiometer and Dial Gauge

Fig. 19: Omega type sensor

Fig. 20: Fixing of anchors plate assembly

Fig. (21): Prestressed carbon laminates

Fig. (22): Epoxy paint marking on laminates

CONCLUSIONS

Post strengthening test indicated that the slabs from 2nd to 5th floor were performing in the expected manner. Also, the corrections incorporated in the design and details of slabs in 6th floor and above did not exhibit any sign of distress. The specification / method of strengthening adopted was less cumbersome as compare to the other specification / methods available. The carbon laminates are now protected against any damage with 2″ thick rich 1:3 cement mortar. The building is commissioned for use.

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ACKNOWLEDGMENTS

Diagnosis, Planning, evolving methodology and execution of retrofitting work was supported by academic establishments, material manufacturers, contactors, and the following persons, whose help is gratefully acknowledged: Dr. R. S. Jangid of IIT- Bombay, Dr. A. N. Bambole of VJTI- Mumbai and contractor R&M International Pvt. Ltd. The author also thanks Dr. Gopal L. Rai for valuable comments and assistance with the design. Covered under the confidentiality, non compete and non conflict clause, names of owners of the project cannot be revealed but the author is thankful to them for entrusting the responsibility of retrofitting work.

A. K. Singh

A. K. Singh is senior faculty in the Department of Civil Engineering at Shree Vile Parle Kelavani Mandal's Shree Bhagubhai Mafatlal Polytechnic, Vile Parle (W), Mumbai. He is an accomplished Educationalist, Structural Designer and Repair and Rehabilitation Consultant, with over 25 years experience in the field of civil engineering and many prestigious projects to his credit.

Mr. Singh is the founder head of A. K. Associates, a professional civil engineering firm involved in catering the need of any kind of Civil Engineering Structures. The aim of the firm is to assure the quality, to assess and diagnose the sick building in scientific and technical manner, to create and sustain the atmosphere of correct Engineering practice to develop competency of Technical staff through Continuous Education and inculcate right Attitude, Approach & Behavior and above all to earn the approval and appreciation from Society.

The firm offers Engineering Services like Total Repair-Rehabilitation Consultancy, Specialized Retrofitting of Existing Structures, Stability checking of buildings, Structural Audit, Inspection and Survey of Building for loss of Durability, Project Management Services, In-house need based Training Programs for working professionals, etc. Prestigious projects successfully completed under his consultancy are intensive retrofitting of N. M. College of Commerce, Bhaidas Auditorium, Jaslok Hospital, Twin Towers, Change of user of theatre building of Ambar-Oscar to Shoppers Stop, dilapidated multistoried building of Syndicate Bank, ESIC Hospitals at Andheri, Parel and Vashi, Intensive retrofitting of Lube Refinery Structure of HPCL, and many more.

PRELIMINARY STUDIES WITH A FLUOROPOLYMER FOR USE IN MODIFIED CEMENT MORTAR FOR WATERPROOFING

Ajay Krishnan, Priya S. Nair, Ravindra Gettu and R. Dhamodharan
Department of Civil Engineering, IIT Madras

1. INTRODUCTION

Waterproofing agents are employed as concrete admixtures to improve the durability of new and/or existing cement concrete structures. There are many commercially available waterproofing agents amongst which polymer modified cement (PMC) mortars seem to offer more cost-effective solutions than other types such as resin and silicone based systems [1]. The advantages of PMC mortars as repair materials include their good bonding to the old concrete, decrease in water/cement ratio (w/c) needed for a workable mix or increase in the workability leading to higher strength and lower permeability, and lower shrinkage cracking [2, 3]. They act by making the concrete surface less permeable to moisture, carbon dioxide and external chemicals [4, 5].

In the work described in this paper, the performance of a fluoropolymer modified cement mortar is compared with the performance with unmodified cement mortar and mortars modified with various waterproofing materials (WPMs) such as SBR, acrylic and integral waterproofing agent. Fluropolymers are widely used for water and oil repellence, and these chemicals possess good resistance to degradation when exposed to UV light and accelerated weathering. Asahiguard E 400 (AG-400) is a commercially available fluorocarbon based product used as water repellant in the fabric industry. It is a perfluroalkylacrylate copolymer, with a chemically structure similar to an acrylic resin. In this study, the AG-400 was added to cement mortar (denoted as FL-ASAH) as a modifier and the influence on the properties of the cement mortar was evaluated.

Fluoropolymers are widely employed due to their resistance against deterioration when they are exposed to weathering conditions, high temperature and fluids. Polymers with fluorinated functional groups could be very useful for repair of concrete structures exposed to sea water or bridges since they are water repellent and prevent adherence of marine organisms [6, 7] on to the repaired surface. Moreover, the fluorine atoms in the waterproofing admixture employed in this work forms a bond with the calcium ions in the cement particles thereby preventing the calcium ions from leaching when exposed to moisture.

2. EXPERIMENTAL PROGRAM

2.1 Materials

The cement utilized to prepare the reference mortar and the WPM modified mortars was a 53 grade ordinary portland cement (OPC) corresponding to Indian Standards. The details of the WPMs used are given in Table 1.

Table 1: Details of the waterproofing agents used

Notation	Type of material	Specifications given by the supplier
N-SBR	Styrene butadiene rubber latex	Milky white liquid, specific gravity: 1.01 at 20 °C
N-AR	Acrylic	Acrylic emulsion, flash point: 480 °C, alkaline, 43% solids
F-IWP	Lignosulphonate	Dark brown liquid
F-ASAH	Fluoropolymer	Milky white, acidic, specific gravity: 1.0 at 20 ºC

2.2 Fabrication

The ratio of cement to sand for the cement mortar was maintained as 1:3, by weight, in all cases. The WPM was incorporated as per the dosage specified by the corresponding manufacturer. Since most of the WPMs were latexes and emulsions in water they enhanced the workability of the mix. In order to make the comparisons objective, mortars were made with the same workability, which is the field condition where water is often added until the applicator gets the consistency needed. In the reference sample (UMM-OPC; without any WPM), the water cement ratio (w/c) adopted was 0.50 and the corresponding flow table spread was 20 cm. In the WPM modified mortars, the WPM was added and the water content was adjusted to give the same flow. The w/c values for each mortar are given in Table 2.

Table 2: WPM dosage and w/c for different mortars having the same flow

Mortar designation	Description	WPM dosage for 50 kg of cement	w/c for flow table spread of 20 cm
UMM-OPC	Unmodified cement mortar	None	0.50
N-SBR	SBR based cement mortars	9 litres	0.37
N-AR	Acrylic based cement mortars	10 litres	0.37
F-IWP	IWP based cement mortars	200 ml	0.48
F-ASAH	Fluoropolymer based cement mortars	5 kg	0.39

2.3 Test procedures

The properties studied to evaluate the performance of the various cement mortars after weathering were the compressive and flexural strengths, as well as the bond strength between the WPM modified mortar and concrete, and water permeability. In all cases, the specimens were demoulded after 24 hours of casting and moist-cured for 7 days (which corresponds to most practical conditions). After this 8-day period, specimens were exposed to weathering or kept in a mist room (i.e., no drying or weathering). Weathering was done in a chamber with UV lamps of 340 nm wavelength and an irradiance level of 1.55 W/m2/nm (to replicate the daylight spectrum). The specimens were subjected to cycles of 8 hours exposure to UV light and 4 hours exposure to a relative humidity of 90%, following the ASTM G-154A standard [8], for 30 days. The compression tests were performed on cubical specimens of 50×50×50 mm following the ASTM C109/C109M-08 standard [9]. The modulus of rupture was evaluated on specimens of 40×40×160 mm in accordance with the ASTM C348-08 standard [10]. Surface water permeability was determined using the Germann water permeability testing (GWT) apparatus conforming to EN-ISO 7031 (1983) [11]. The bond strength between the repair material and concrete was evaluated using pull-off tests performed according to the ASTM C 1583/C 1583M-04e1 standard [12]. The shrinkage strain was evaluated according to ASTM C1148-92a [13] and the capillary absorption tests were conducted according to RILEM recommendation CPC11.2 [14].

3. RESULTS AND DISCUSSION

In all the results shown here, the following notations are employed: "Control" indicates that the specimen was in ambient conditions (i.e., no weathering) for 28 days; "Weathering" indicates that the specimen has undergone 30 days of weathering (after 8 days of moist curing) as described earlier; "Constant flow" refers to specimens prepared with WPM modified cement mortar where all have the same workability.

3.1 Compressive Strength

It can be seen in Figure 1 that weathering decreases the strength substantially in the case of the OPC and the F-IWP mortars. Interestingly, the strength increases for the mortar with the SBR latex when subjected to accelerated weathering conditions. It is observed that the decrease in compressive strength due to weathering of the unmodified mortar (26%) and the F-IWP (43%) mortar is higher than in the case of the other polymer modified mortars. The higher values of compressive strength of the latex based mortar after weathering may probably be due to the bridges and plugs formed by the polymer in the cement microstructure [4]. In the case of the fluoropolymer based F-ASAH mortar, the decrease in strength (about 10%) after accelerated weathering is slightly less than that of the reference mortar.

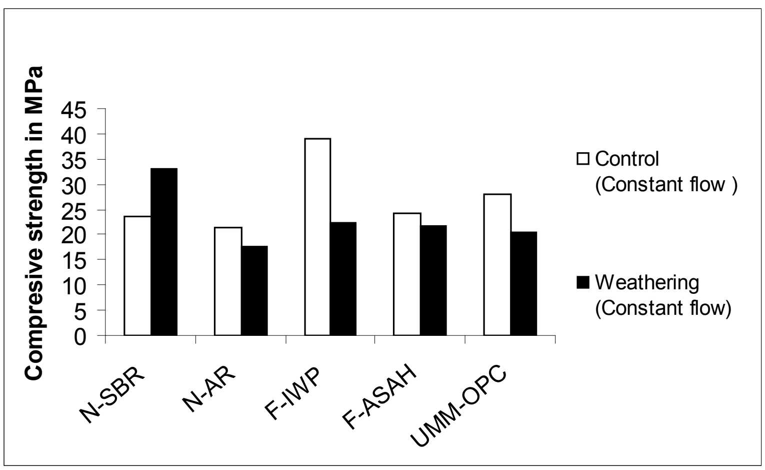

Figure 1. Compressive strengths of the mortars before and after weathering

3.2 Flexural Strength

The values obtained for flexural strength (see Figure 2) follow a trend similar to that of the compressive strength. The decrease in flexural strength after weathering is about 5% in the IWP mortar and about 8% in the case of the unmodified cement mortar. It can also be seen that the addition of latex increases the resistance to cracking even after weathering; the strength of the latex based cement mortars after weathering is about 1.5 times that of the control mortar. This confirms that there is an increase in the resistance to deformation under stress after weathering when a latex emulsion is added to cement mortar [15]. A significant increase is also observed for the F-ASAH samples though not as much as the latexes.

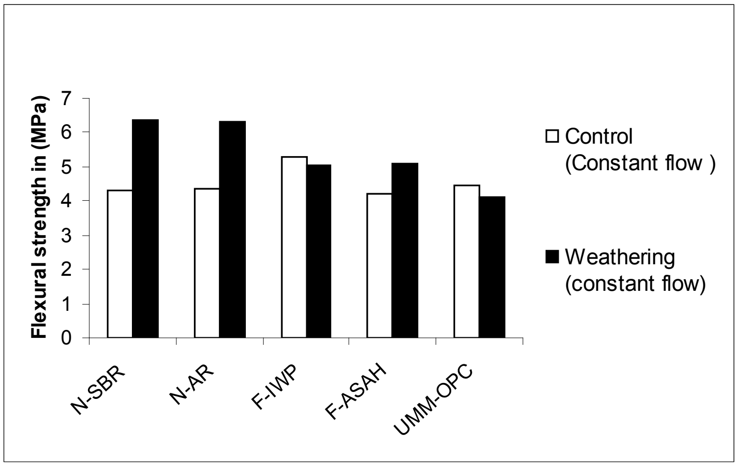

Figure 2. Flexural strengths of the mortars before and after weathering

3.3 Water Permeability

Figure 3 shows that the coefficient of water permeability (Cp) increases in both unmodified and WPM modified cement mortars after exposure to accelerated weathering. On weathering, specimens of UMM-OPC, N-AR and F-IWP mortars exhibited a drastic increase in permeability compared to the control specimens. On the other hand, it was observed that the increase is low for the SBR latex and fluoropolymer based cement mortars. This is possibly because the SBR is generally resistant to degradation on exposure to UV radiation and, therefore, the partial filling of micropores and voids by the polymer continues to prevent the ingress of water [16]. The F-ASAH mortar exhibited the lowest water permeability even after exposure to accelerated weathering. In the unmodified cement mortars, the permeability increases to a greater extent due to the damage caused by the stresses induced by the alternate wet and dry cycles during weathering. It was observed that no value could be reported for the UMM-OPC mortar subjected to weathering since it was highly permeable.

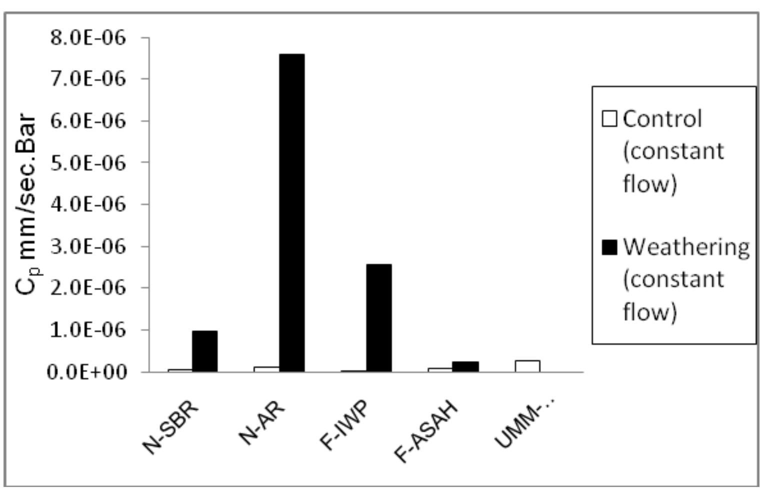

Figure 3. Water permeability of the mortars before and after weathering

Since the fluoropolymer is hydrophobic, the permeability of water through the cement matrix having this polymer becomes lower than the other mortars studied. The cement mortars with this material were almost impermeable to moisture even after UV exposure and weathering. The fluoropolymer modified cement mortar also exhibits superior bond strength before and after UV exposure and wet and dry cycles. This may be attributed to the resistance of the fluoropolymer against degradation when exposed to UV light and moisture, which enables the retention of bond strength even after weathering [17]. The above data indicate that such mortars could be employed for waterproofing under exposed conditions.

3.4 Capillary absorption

Capillary absorption of water by the cement mortar is also an important criterion to evaluate the performance of WPM modified mortar. From Figure 4, it can be seen that there is a considerable reduction in capillary absorption for the SBR modified cement mortars compared to the reference specimens. Among the WPM modified cement mortars, IWP and SBR modified mortars gave better reduction in capillary water absorption than the AR modified mortar, in which the capillary absorption of water was found to be the highest. The F-ASAH mortar exhibited the least capillary absorption.

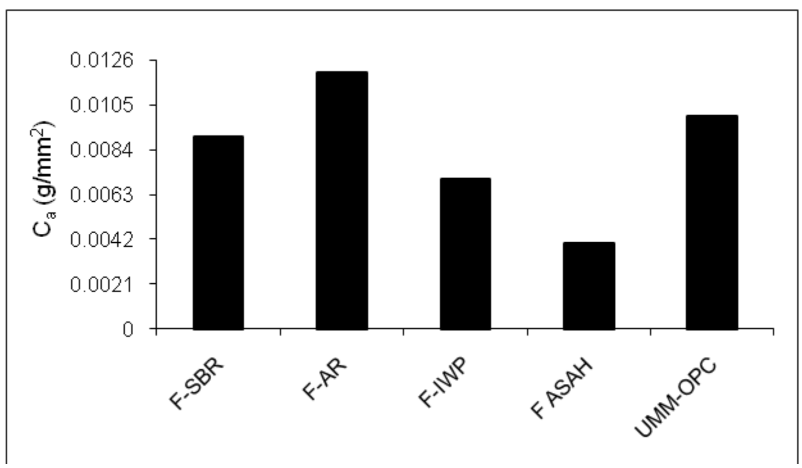

Figure 4. Capillary absorption of unmodified and WPM modified cement mortars

3.5 Shrinkage strains

Shrinkage is one of the most important parameters to be investigated in case of repair materials since it can result in cracking and, subsequently, increase the permeability and limit the durability of the repair work. The early shrinkage strains in the different mortars were assessed and are reported in Figure 5.

In all the WPM modified mortars, there is a reduction in shrinkage strain of more than 75% due to the addition of the waterproofing agent, compared to the reference mortar (UMM-OPC) after 10 days of drying. This can be attributed, in addition to the film formation, to the increase in the viscosity of the pore solution in cementitious composite due to the addition of the polymer latex. This, in turn, leads to reduction in the capillary tension and reduction in shrinkage [18] of the cement mortar. Such reduction in shrinkage strain is one of the most important factors in determining the suitability of these admixtures for waterproofing applications. Similar to the latex based mortars, the F-ASAH mortar exhibited low shrinkage compared to the reference mortar.

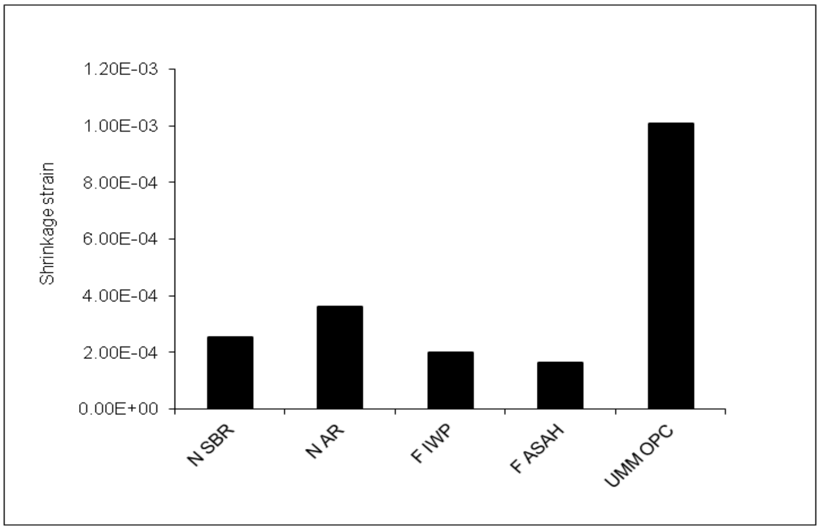

Figure 5. Shrinkage strain at 10th day of drying

3.6 Bond Strength

Figure 6 illustrates the decrease of bond strength in all the mortars on exposure to UV light, and alternate wet and dry cycles, though the SBR based mortar showed better strength than the OPC and other WPM cement mortars. The results also confirm that better bonding can be obtained by the incorporation of latex even after weathering. The mortars containing the acrylic had higher bond strength than the plain cement mortars but not as high as the mortar with the SBR latex. F-ASAH based mortars exhibited good bond strength compared to reference mortar, even after weathering.

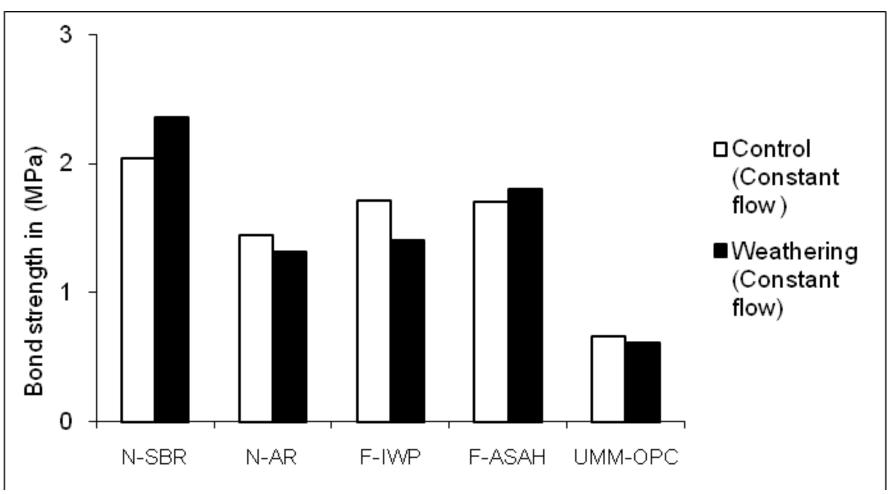

Figure 6: Bond strengths of the mortars before and after weathering.

4. CONCLUSIONS

- The use of a fluoropolymer for modifying the cement mortar for the purpose of waterproofing has been evaluated by adopting the same criteria as for the other commercially available WPMs. This material exhibited better resistance to deterioration on exposure to weathering conditions.
- The fluoropolymer also yielded better water repellent characteristics and the least water permeability among all the WPMs tested.
- Further research is needed for determining the optimum fluoropolymer dosage and cost-effectiveness.

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Ravindra Gettu

Ravindra Gettu is a Professor of Civil Engineering at IIT Madras since 2004. His research interests are in the effective use of chemical admixtures, fibre reinforced concrete, characterization of the nonlinear response of materials and housing.

REHABILITATION OF A TSUNAMI-DAMAGED LIGHTHOUSE AT LITTLE ANDAMAN

Devdas Menon

Abstract: The reinforced concrete (RC) lighthouse at Little Andaman Island, built during 1973-1980, was struck by the tsunami/earthquake on 26 December 2004. This 45m high cylindrical tower was badly damaged in the lower portion, with extensive spalling and crushing of concrete and buckling of the vertical reinforcement on one side, and yielding and fracturing of reinforcement on the other side (facing the sea), along with severe damage and tilting of the RC portico structure connected to the tower at the entrance. The Department of Lighthouses and Lightships, Government of India, sought the assistance of IIT Madras to explore the possibility of salvaging this lighthouse, as an alternative to the expensive option (recommended by an expert committee) of demolishing and rebuilding the structure.

The presentation describes the details of the rehabilitation measures proposed by IIT Madras and successfully implemented at site. The remedial measures consisted primarily of concrete jacketing of the lower portion of the shell structure from outside and inside, with additional thickening on the outside (to correct the eccentricity in vertical alignment) and provision of additional vertical rebars, properly anchored to the well cap below and the undamaged portion above. The repaired lighthouse has safely resisted several high intensity tremors in the region, without any visible damage.

Devdas Menon

Devdas Menon is presently a Professor in the Department of Civil Engineering at IIT Madras, engaged in teaching, research and consultancy in structural engineering, and in developing a holistic approach in education, with emphasis on inner development and transformation.

In engineering, his primary research interests are in the area of structural concrete design, and in the analysis and design of buildings, bridges, towers and chimneys. He has also carried out innovative research and development in affordable and sustainable building systems and in biomechanical orthopaedic devices. He has published a large number of technical papers, and received patents and awards. He has a special interest in developing codes of practice, and is presently the Chairman of the Bureau of Indian Standards CED 38 Committee on "Special Structures".

He has a deep interest in teaching and continuing education, and has authored / co-authored several textbooks, titled "Reinforced Concrete Design", "Structural Analysis", "Advanced Structural Analysis" and "Handbook on Seismic Retrofit of Buildings".

Devdas Menon has also authored a book called, "Stop sleepwalking through life!" (1998), and has been conducting numerous lectures and workshops for students, teachers and corporate organizations, on finding meaning and fulfilment in life through self awareness and inner transformation. He teaches a uniquely designed elective course at IIT Madras called "Self Awareness", which is open to all interested students and faculty.

He was recently awarded the "Distinguished Service to the Institute Award (2013)" for his contributions to the legacy and future of IIT Madras by the IIT Madras Alumni Association.

On Association of Structural Rehabilitation and few Interesting Techniques for Retrofitting

G. R. Reddy

Abstract: One of the organizations where public are generally served better is social society. One of the societies is Association of Structural Rehabilitation (ASTR). Objectives of ASTR a nongovernmental, non profitable and social organization are discussed in brief. To make the paper complete, some of the interesting techniques for economical design and retrofitting of structures considering external natural events are also discussed.

Association of Structural Rehabilitation (ASTR)

The motivations behind forming ASTR are observations of large number of structures collapse every year, even under normal loads. The structure could be of heritage, residence, office, lifelines etc. The problems are more severe in the case of RC structures. This is also true in some new structures. The reasons are mainly ageing effects such as corrosion, carbonation, long term creep and shrinkage. Also other contributors are poor design, material and construction quality, interactions due to new constructions, Ground movements etc. One of the cases shown in Fig.1 is a thirty three year old building in Mumbai developed cracks at the ground level and tilted suddenly to the noticeable level. This may be due to long term creep, shrinkage, corrosion. Ground settlement also cannot be ruled out. The structure would have been in good condition if its health is regularly assessed with suitable instrumentation and repaired/rehabilitated.

Fig.1 Failure of columns in RCC structure

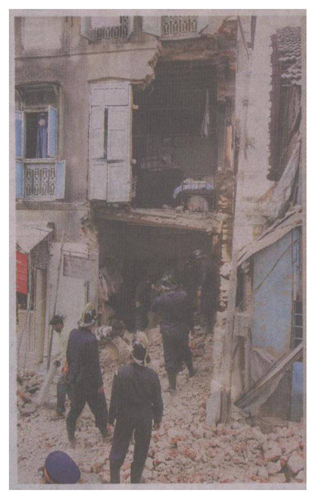


Fig.2 Failure in part of 100 years structure

This eventually would have helped for the safety of the people. Fig.2 shows a part of the 100 years weak building crashed. The failure attributed to the damaging of load bearing member while making modifications in the flour mill area. This may be due to unawareness of importance of structural members and not assessing capacity of deteriorated structural member. Proper health assessment, evaluation of capacity of the structural elements and extra strengthening if required would have been avoided the failure.

In addition to above, the design requirements are becoming stringent and more demanding in the case of earthquakes, cyclones, and Tsunami keep on increasing. If the structures are not attended and retrofitting is not performed wherever necessary, failure of structures unavoidable. There is technology and procedure which makes the structure safe. The difficulty is that the procedure is not transferred to the needy in simple and adoptable way. The public awareness is very poor. Some times it is made assumption that cost of rehabilitation and retrofitting is very expensive. In addition, the technology is also applied in ineffective way.,

To combat such factors it is found necessary to form a social association which will work for the safe structures and eventually for the safety of the people. A group of Engineers of India have formed Indian Association of STructural Rehabilitation (ASTR) which will work towards achieving excellence by delivering engineering services for any kind of civil engineering structures meeting need based rehabilitation requirements. ASTR got registered and some of the objectives to be met by the association are as follows.

1. To cater the need of sick Civil Engineering Structure with Assessment and Diagnosis in a Scientific/Technical manner.
2. Develop competency of technical persons through training and continuous education program to inculcate right attitude, approach and behavior through theme meetings, workshops, conferences etc.
3. Share the knowledge, expertise and experience of structural rehabilitation through publications such as news letters, manuals, standards etc.
4. To see the procedures reach builders, contractors, designers including normal public
5. Develop local chapters for quick promotion and establish easy approach to needy.

To fulfill the objectives, research community, designer community, industrial/application community and administrative community need to join the hands. There is a large gap among these communities. The applied research should be need based and researchers need to understand the industrial/application community requirements. There should be good interactions between the designer and the researcher to see the new research work is completely understood and applied through industry and administration. In simple the technology go on cyclic process. In this closed chain the public plays very important role and their participation shall be given high priority.

Already large number of personnel of various communities has become the members of the association and more are in pipeline to become members to serve the association. Four local chapters such as Mumbai, Goa, Mangalore and Bangalore chapters are formed and efforts are continued to start more local chapters.

The following sections discuss the second part of the article.

General Steps in Rehabilitation and Retrofitting

1. Formation of teams – Comprising Owner/utility, designer, analyst, NDT expert, regulator, expert in rehabilitation/retrofitting.
2. Deriving demands such as dead loads (include design intended loads), earthquakes, wind as per the current practice.
3. Gathering design and quality assurance report, information on type of structures, type of construction etc.

4. Assessing the information on in-situ condition of the structure (Non-Destructive Examination).
5. Assessment of capacity of the structure with respect to the derived higher or design intended loads as per the current design practice.
6. Developing and designing of rehabilitation scheme and implementation.

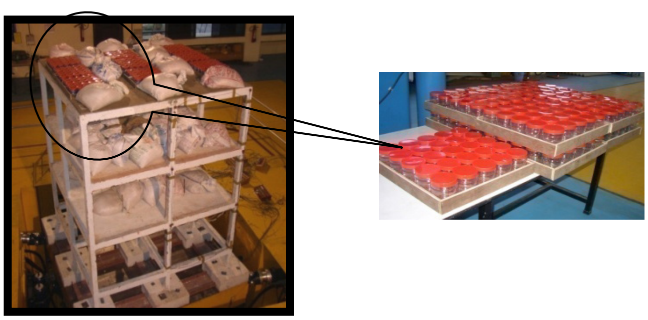
Interesting design/retrofitting techniques for structures subjected to external loads

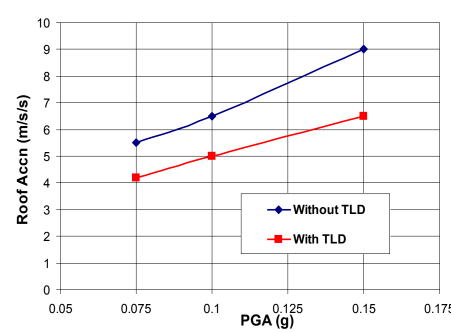
There is large number of acceptable passive control methods to reduce the dynamic response of structure subjected to earthquake or wind loads. These techniques can be adopted during design stage or can also be used for retrofitting existing building. Two of those techniques are tuned liquid dampers and bas isolation. Brief description of these techniques is given below.

Tuned mass liquid dampers

Large number of tall structures as shown in Fig.3 is being built in cities. The design of tall buildings is governed by wind loads. Tuned Liquid dampers (TLD), tuned mass dampers, pendulum dampers are generally considered in the design and implemented. However, if the tall structures are designed with out such devices and if there are some sign of wind induced oscillations and cracks are being developed in the structure, one can use these devices for retrofitting. A TLD consists of a rigid tank with shallow water and rigidly connected to the

Figure 3: Typical Tall Buildings





(a) RCC model with TLDs

(b) Variation of response with/without TLDs

Figure: 4 Effect of Tuned Liquid Dampers

performance of such devices was shown experimentally and results are given in Fig. 4(b). The structure was exited with base motion with varying Peak Ground Acceleration (PGA). It is clearly seen that the response reduces significantly when TLD s are used. Effectiveness of TLD is enhanced by using ferro-nano fluid in place of normal fluid because ferro nano-fluids exhibit enhanced viscosity under externally applied magnetic field. Shake table tests were conducted using normal TLD and TLD with ferro nano-fluids of different concentrations with and without magnetic field in gauss units as shown in Fig.5. Experiments were also conducted at different levels of magnetic filed strengths. The effectiveness of normal TLD and TLD with ferro nano-fluid with and without magnetic field can be observed from Fig. 6 (a) & (b) respectively.

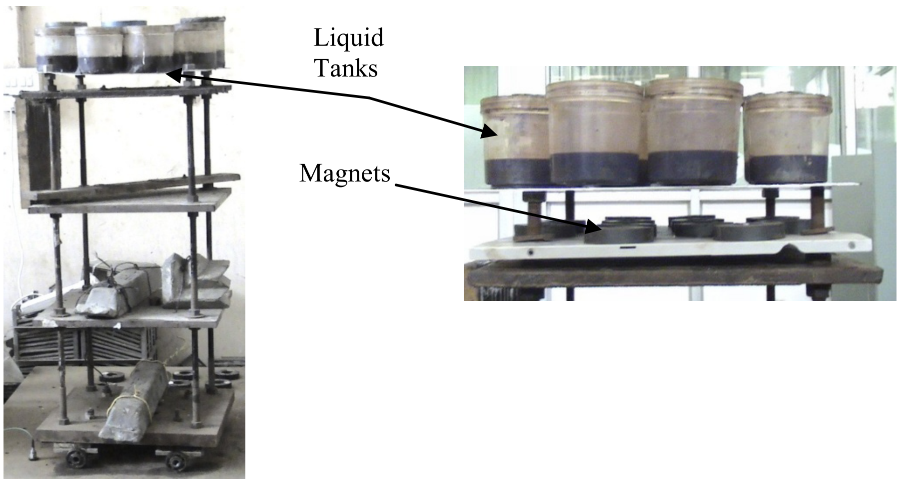


Figure: 5 (a) Tuned liquid Damper model (b) TLD with Ferro nano-fluid

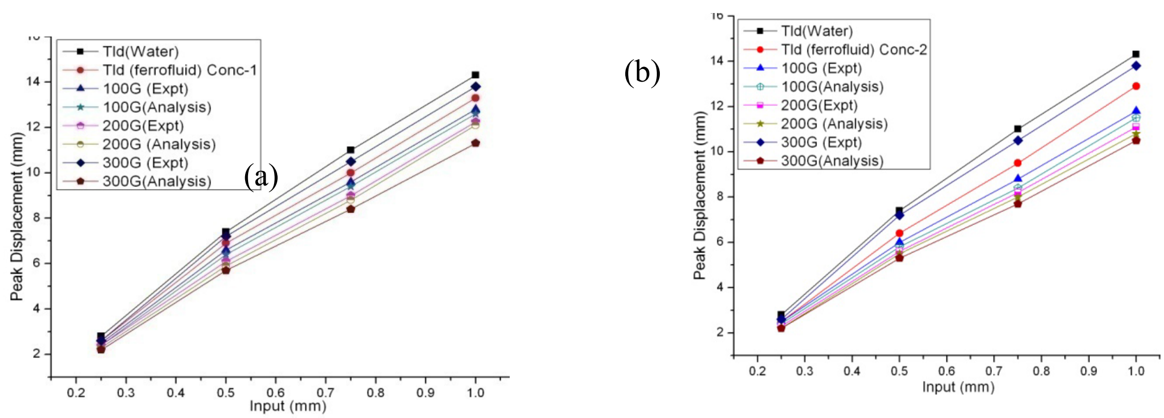
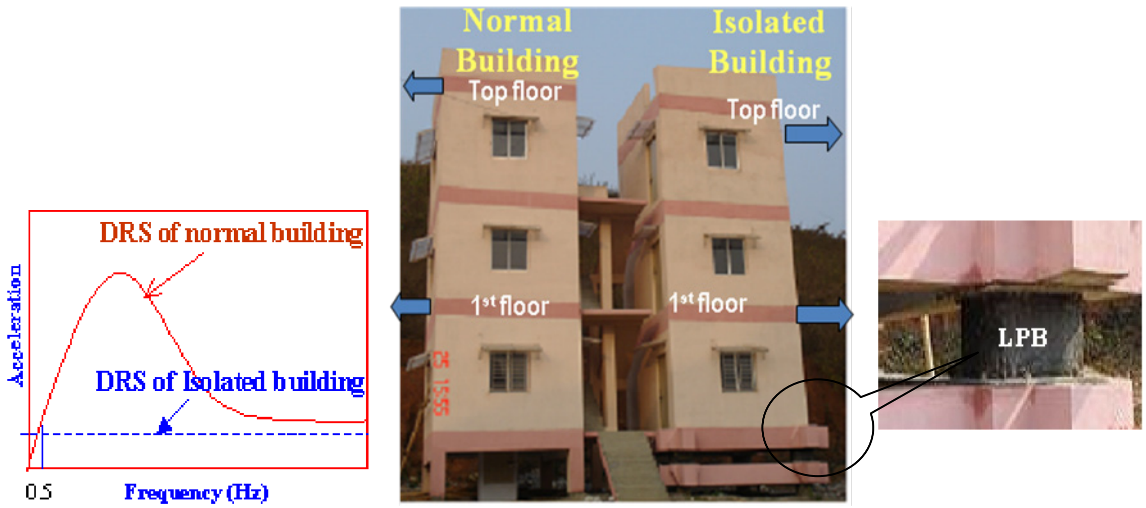


Figure: 6. Response of Structure using TLD with (a) ferro-fluid of 4% concentration (b) ferro-fluid of 8% concentration

Seismic Base Isolation

The base isolation is aimed to attenuate the horizontal accelerations transmitted to the superstructures. The isolators attempt to decouple the building or structure from the horizontal components of the ground motion. Isolators have low horizontal stiffness and they are placed between the structure and foundation as shown in Fig.7. This makes the natural frequency of the whole system including the structure to the order of 0.5 Hz (2 sec period) or less as shown in Fig.7(a). This eventually makes the structure to experience low accelerations which is shown in terms of Design Response Spectrum (DRS) for normal building and DRS for isolated building. Thus the structure along with the isolators acts like filter and does not respond to higher frequencies (> 0.5Hz). This is analogues to a parked ship subjected to wave motion causing rigid body motion of the ship without causing loads on it.

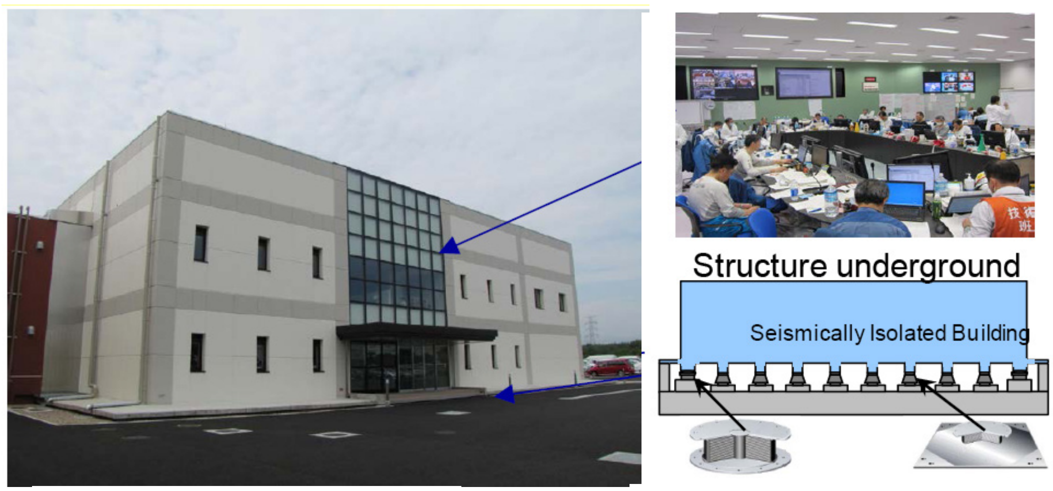


(a) Concepts of Base Isolation (b) Normal and isolated buildings at IIT Guwahati. On the left is the close-up of lead plug bearing (LPB)

Figure: 7: Concepts of Base Isolation

Performance of seismic base isolated emergency response centers

In the IAEA international workshop on earthquake preparedness and response for NPP organized by International Atomic Energy Agency along with State Nuclear Power Technology Corporation, China from 24-28, October, 2011, participants from Tokyo Electric Company mentioned that with the experience at Kashiwajaki-Kariwa nuclear power plant due to Chuetsu-oki earthquake occurred on 16 July 2007 around Niigata Prefecture, an emergency response centre was built at Fukushima-Daiichi NPP station as shown in Fig.8(a). The structure is two storied RCC with total floor area of 3700 m^2. It is supported on seismic base isolators as shown in Fig.8 (b). It housed communication equipment such as exclusive telephone lines between NPP and headquarters office and government agencies. It also house power supply equipment such as gas turbine driven generator. This facility played very important role after the 11th March, 2011 accident. Plant manager and responsible staff stayed there and managed the restoration activities. It helped to have communication with Government officials, Head quarter official through TV communication system.



(a) Emergency Response Centre (b) Base isolators in Foundation Response Centre

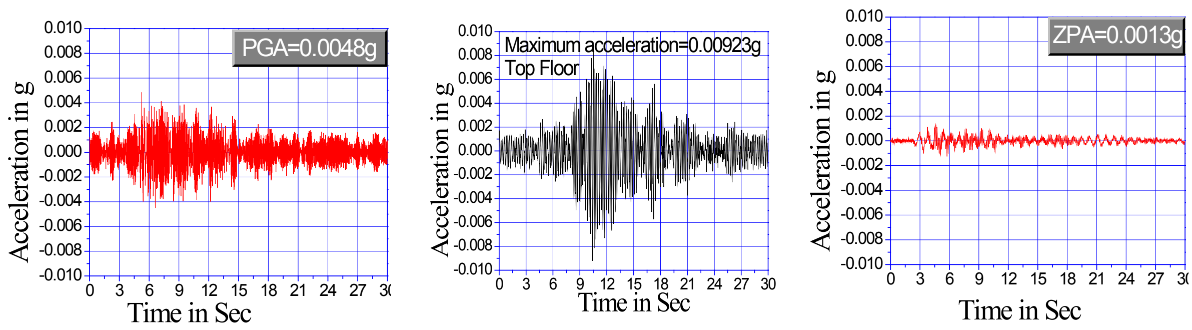
Figure 8: Emergency Response Centre Supported on Base Isolators at Fukushima-Daiichi NPP

Performance of base isolated RCC framed building under actual earthquake

To study the effectiveness of base isolation, two numbers of three-storied framed RCC buildings with similar construction, one building with conventional foundation (here onwards called as normal building) and other with base isolation as shown in Fig.7, were built at Indian Institute of Technology (IIT), Guwahati campus. Guwahati is situated in the most severe seismic zone (Zone-V) of Northeast India. Northeast India is lying at the juncture of Himalayan Arc to the north and Burmese Arc to the east, and is one of the most active regions of the world. Eighteen large earthquakes ($M \geq 7.0$) including two great earthquakes ($M \geq 8.0$) occurred in this region during the last 100 years. For base isolation Lead Plug Bearings (LPB) of 50T vertical load capacity have been used under each column of the building.

A twelve channel dynamic structural recording system has been employed for recording the seismic ground motion and structural response of normal and isolated buildings. One tri-axial accelerometer has been installed on the ground to capture earthquake induced ground motion and four numbers of accelerometers have been installed at 1st floor and 3rd floor of the buildings to record the bi-directional accelerations.

Fig.9 shows the response of two building recorded during Nov 06, 2006 earthquake: Magnitude 5.2, Location: 24.736^{o} N 95.223^{o} E. It can be clearly seen that there is more than two times amplification of ground acceleration at top of normal building, whereas maximum response of isolated building is even less than peak ground acceleration. There is two to five times reduction in response from 1st to top floor of isolated building as compared to the normal building, which shows the effectiveness of base isolation in seismic response control. The recorded response of the base isolated building is almost constant along the height of the building, which also depicts the rigid body motion of the base isolated building. Due to this behavior, the margins in isolated buildings will be very large and can easily withstand beyond design basis events provided sufficient margins are built in isolators which is fairly easy task. This strategy makes the structure less vulnerable for earthquake loading.



(a)Ground motion (b) Conventional building third floor response (c) Isolated building third floor response

Figure 9: Seismic performance of conventional and base isolated structures

Green technology for Seismic base isolation

Seismic isolation of structures can also be obtained using natural materials such as sand and non-degradable materials such as plastics. Research was initiated along with National Institute of Technology (NIT), Surathkal and Central Power Research Institute (CPRI), Bangalore. Large number shake table tests were performed and found that natural materials below the foundations and hard strata which greatly help to control the seismic response of structures. The hard strata can be created naturally or artificially depends on the local plant site characteristics. Clean river sand was tried as base isolation material and experiments were conducted on a RCC frame model having raft foundation as shown in Fig.10 (a) and (b). The size of the model is 1.2 x 1.2 x 1.5 m and the model is provided with a base raft of size 1.5 x 1.5 x 0.1 m thickness. The size of the cross section of beams and columns is of 0.1 x 0.1 m. and roof slab thickness is 0.05 m. A metallic box of size 2.0 m x 2.0 m x 1.2 m height as shown in Fig.3 is used and is filled with 300 mm thick layer of the dry sand / wet sand / dry sand with geo-membrane / wet sand with geo-membrane in separate cases. The box is fixed on tri-axial shake table and RCC model is kept on the sand layer. The natural frequency of the structure was found to be 4.5 Hz along horizontal direction which has reduced to 3.3 Hz in the case of wet sand layer. This effect is shown in Fig.10 (d) which gives variation of Fourier amplitude with frequency.. The damping of the system has also increased from 6% to 12 % due to soil layer when the structure was excited with sine wave. Due to improved dynamic characteristics the response acceleration recorded at the top of the structure is reduced from 7.5 m/sec^2 to 5.1 m/sec^2 as shown in Fig.10(c). This shows about 30 % of reduction and can be concluded that this is also one of the viable solution to isolate the ground motion reaching to the structures.

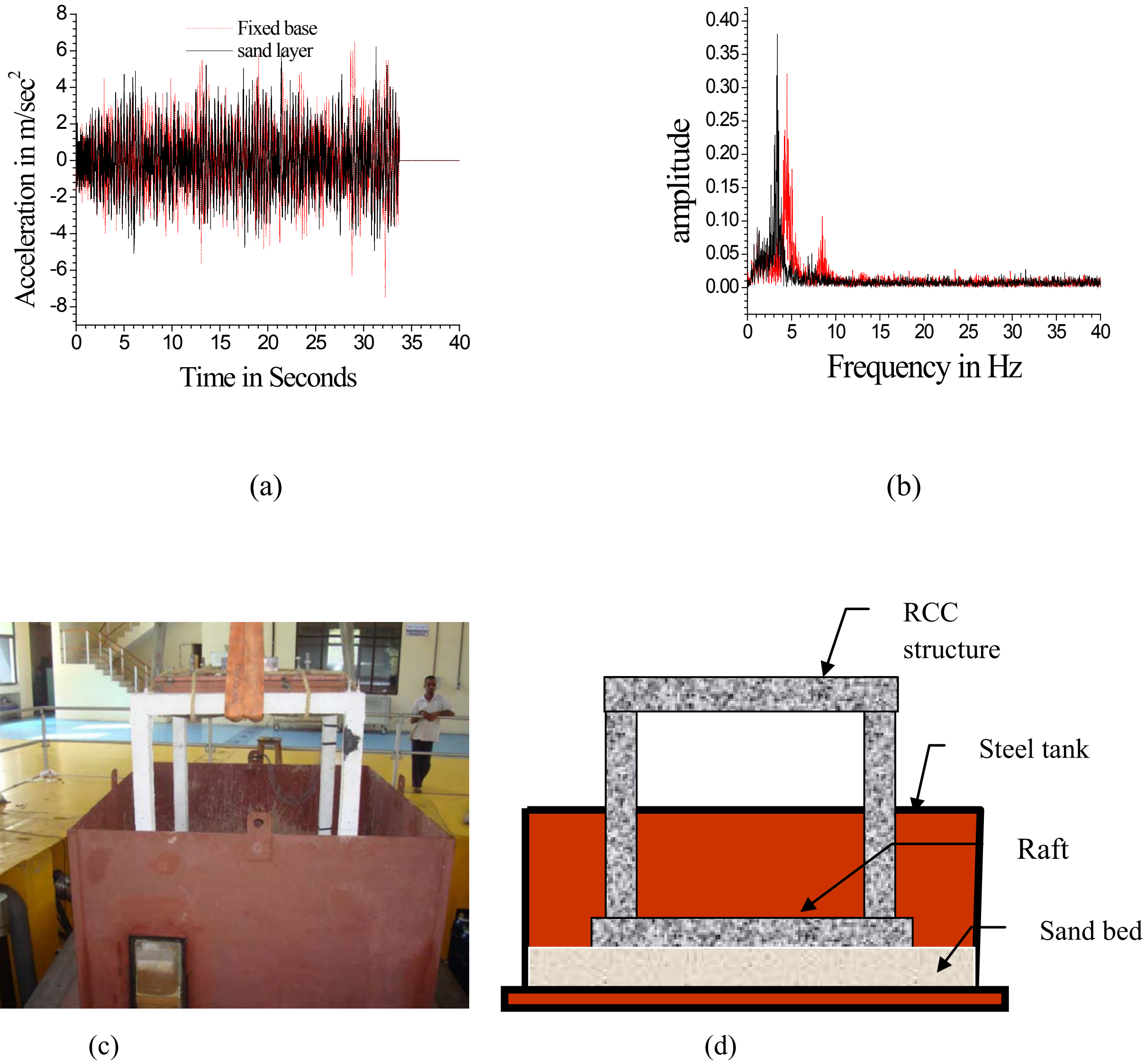


Figure 10: Seismic behavior of structure supported on sand layer

Discussions

Need and objectves of ASTR are briefly discussed. Efforts are made to explain the importance of health of structures for the public safety. To make the paper complete, usefulness of tuned liquid dampers and base isolation for controling dynamic response is explaiend considering exerimental data. Also briefly explianed how the base isolated emergency response centre helped to manage the eartquake disaster during 2011 Fukushim seismic event.

Acknowledgements

Large number of researchers from IITG, Guwahati, NITK, Surathkal, CPRI, Bangalore, BARC, Mumbai etc., were involved in the work presented briefly in the article. The author expresses sinsere thanks to all of them.

G. R. Reddy

Professor G. R. Reddy, B.E, M.Tech, PhD(Japan) is heading Structural and Seismic Engineering section of Reactor Safety Division of Bhabha Atomic Research Centre and also serving as Professor, Homi Bhabha National Institute. In addition he is a recognized guide for PhDs in Mumbai University. He is holding honorary post of president of Association of Structural Rehabilitation (ASTR). Since 1984 he is working in the area of Structural Dynamics & Earthquake Engineering and analyzed, designed nuclear facility structures, equipment and piping systems. Made significant contribution in design of 500 MWe PHWRs and involved in the design of AHWR. Made contribution in the design of 30 M DSN antenna for Chandrayan project and recently completed the design of large size gamma ray telescope. Worked in the research areas like modeling techniques of complex structures, structure-equipment interaction due to earthquakes, stochastic methods of analysis, dynamic substructure techniques. Mastered seismic response control methods and involved in developing friction dampers, elasto-plastic dampers, Lead extrusion dampers, isolators and Tuned liquid dampers. Developed simple seismic design procedures for equipment and piping supported on hysteretic supports. As a part of developing more realistic design procedures, performed large number of experiments on beam-column joints, frames and piping systems till collapse. Simple method to evaluate the performance levels of structures and piping systems is developed. For the purpose of life extension of the existing facilities, evolved methods for performing seismic retrofitting of structures using dampers, FRP, steel jacketing. Working as expert member to support the disaster management activities of Municipal Corporation of Greater Mumbai. He has guided several students for M.Tech and PhD. He has more than 360 publications to his credit.

Structural Restoration of Turbhe Flyover on Sion-Panvel Special Highway

Abhay Bambole

Abstract: The bridge at Turbhe junction of Navi-Mumbai connects Mumbai to Thane and Panvel, construction and opened to traffic in 1994. The bridge has four arms. It has total 59 spans out of which 4 spans are over railway line which are pre-stressed and remaining are 55 RCC. The span length of all the individual spans is about 30m. Turbhe bridge started showing distresses in the form of pot-holes in the deck slab within 3 years of the construction.

The superstructure (RCC Bridge Girders) has sagged / deformation due to long term effects of sustained loads and higher vehicular loads. Superstructure design is adequate for the intended live loads (vehicular loads as per IRC guidelines). Higher axle loads lead to localized damage of deck slab in turn giving rise to pat-holes. Pot-holes developed in the deck slab lead to loss of overall stiffness of the girder. Loss of stiffness of girders due severally damaged (pot-holes) deck slab induced multiple flexural and shear cracks in the webs and bottom slab of RCC girders.

The deck slab was repaired by an RCC overlay of 100 mm on deck slab with appropriate shear connectors to ensure monolithic action between old concrete and new concrete. The additional load induced by concrete overlay was within the design limits. Further, the bridge needs to be strengthened for the vehicular loads (heavier vehicles) subjected during traffic congestion. FRPC repair is a simple way to increase both the strength and design life of a structure. Because of its high strength to weight ratio and resistance to corrosion, this repair method is ideal for concrete structures deteriorated due to increased lads and environmental factors. It has been observed that, by encasing concrete members, FRPC protects existing steel and concrete from deleterious effects. Detailed analysis and design of strengthening system using CFRP (Carbon Fibre Reinforced Plastic) plates and layers of FRP wraps have been carried out as per the guidelines of basis of Technical Report no.55 of Concrete Society, UK. Comparative studies of FRP strengthening and steel plate strengthening shows that the later scheme is not only higher in cost but also induces significant dead weight to structure.

ABHAY BAMBOLE

Dr. Abhay Bambole is presently the Professor and Head of the Structural Engineering Department, VJTI, Mumbai. He has 19 years of UG and PG students teaching experience. He completed his PhD at IIT Bombay in 2008. His research interest includes Composite Mechanics, Finite Element Method, Non-Destructive evaluation techniques, Fibre Reinforced Composites in Repair, Retrofitting and Strengthening of structures, Fire Endurance of RC structures, Waterproofing of RC structures. He published 3 journal, 6 conference publications with 1 Books/Proceedings. He is also keen in disseminating technical knowledge through lectures (20) and very enthusiastic in learning new concepts/ techniques by attending seminars/courses (25). Currently he is working is various projects involving FRP wraps and composite plates. He is member of many national and international societies (ASCE, ISTE, INSDAG, ICRI, ISNT, ASTR), Technical Committees (High Rise Buildings, Mumbai Heritage Conservation Committee), Member (elected) on Board of Management of College as per Statute for autonomous institutes (2010-2012). He also made significant contribution to the admission process for First year Engineering students, Maharashtra State.

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