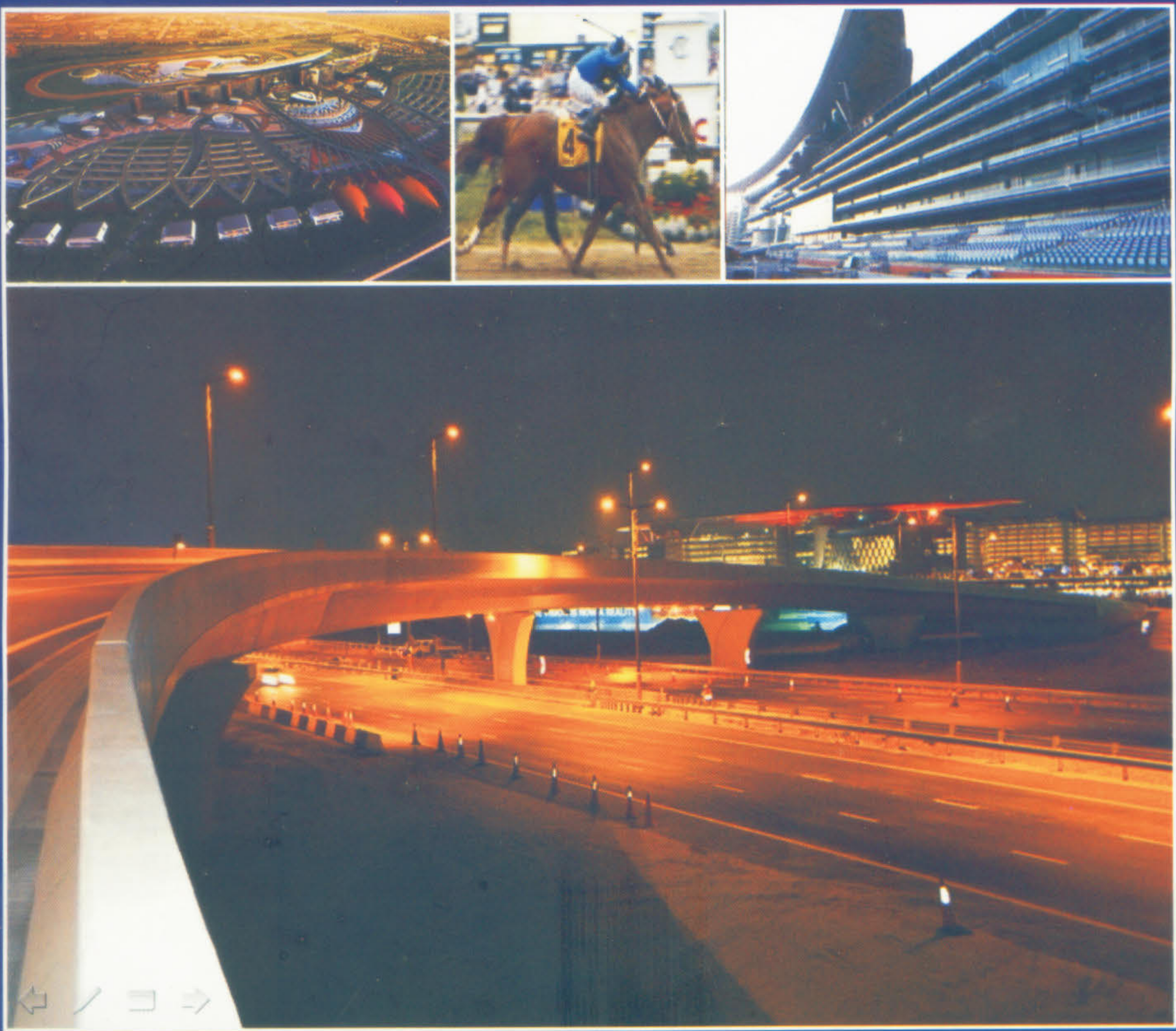
STRUCTURAL ENGINEERING

QUARTERLY JOURNAL OF
INDIAN SOCIETY
OF
STRUCTURAL ENGINEERS

ISSE

VOLUME 13-2

Apr-May-Jun-2011



NAD-AL-SHEBA RACE COURSE DEVELOPMENT PROJECT : CONSTRUCTION OF THREE BRIDGES

(See page 10 inside)

LET US BUILD A STRONG STRUCTURE OF INDIAN SOCIETY

STRUCTURAL ENGINEERING

QUARTERLY JOURNAL OF

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

ISSE

VOLUME 13-2, APR-MAY-JUN 2011

Head Office : C/O. S G Dharmadhikari, 24, Pandit Nivas, 3rd Floor, S K Bole Marg
Dadar (W), Mumbai - 400 028 • Tel. 91-22 24365240 • Fax . 91-22 -2422 4096
• E-mail : issemumbai@gmail.com • Website : www.isse.org.in

Regd. Office : The Maharashtra Executor & Trustee Co. Ltd., Bank of Maharashtra, Gadkari Chowk
Gokhale Road (N), Dadar, Mumbai - 400 028

Charity Commissioner Reg. No. E 17940, Mumbai   Donations are exempted from Income Tax under 80-G

Contents

- Fraternity News 2
- Conceptual design issues in earthquake engineering
  By Andrew Charleson 3
- Report on ISSE lecture on "FM Global's approach to safe design practice to limit property losses" 11th May 2011 9
- NAD-AL-SHEBA race course development project : Construction of three bridges
  By Vivek G. Abhyankar 10
- APPEAL TO ISSE MEMBERS 17
- Understanding of Stress Flow Pattern Using STM & FEM
  By Prasad Samant, Hemant Vadalkar 19
- Report on ISSE One Day Workshop on "Wind and Earthquake Loads" - 21th May 2011 24

Editor : N K Bhattacharyya
Jt. Editor : Hemant Vadalkar

Fraternity News

WELCOME TO NEW MEMBERS

(Apr-May-Jun 2011)

LIFE MEMBERS

M-1086 Mr. Santosh Shantaram Thakare
M-1087 Mr. Siddharth Indranath Bothra
M-1088 Mr. Kiran Sidramappa Patil
M-1089 Mr. Chandrakant Dattatraya Chaudhari
M-1090 Mr. Sunil Sandeep Patil
M-1091 Mrs. Yogita Shankar Sharma
M-1092 Mr. Jaydhaval Ashok Karkamkar
M-1093 Mrs. Gouri Subhash Ghodake
M-1094 Mr. Jawahar Prabhuraj Upase
M-1095 Mr. Madhukar Ambadas Sul
M-1096 Mr. Vishwas Nandkishor Khatri
M-1097 Mr. Anil Ashok Gandhi
M-1098 Mr. Paresh Vijay Kulkarni
M-1099 Mr. Hemant Kumar Vinayak
M-1100 Mr. Sunil Vasudeo Chavan
M-1101 Mr. Anil Govind Shrotri
M-1102 Mr. Pradeep Shantaram Kiwalkar
M-1103 Mr. Sunil Keshav Kulkarni
M-1104 Mr. Shamrao Mohan Raut
M-1105 Mr. Balmukund Ramesh Rathi
M-1106 Mr. Chetan Gourishankar Konapure
M-1107 Mr. Sanjeev Kumar Tyagi

ORGANISATION MEMBER

OM – 19 – Pendharkar Consultancy Pvt.Ltd

REVISED STRENGTH AS ON 30-6-2011

Patrons : 29 Organisation Members : 19 Sponsors : 8

Members : 1107 Junior Members : 9

TOTAL STRENGTH : 1172

FIELDS CONSIDERED AS ASPECTS OF STRUCTURAL ENGINEERING

- Structural Designing & Detailing
- Computer Software
- Materials Technology, Ferrocement
- Teaching, Research & Development
- Rehabilitation of Structures
- Construction Technology & Management
- Geo-Tech & Foundation Engineering
- Environmental Engineering
- Non Destructive Testing
- Bridge Engineering

& Other related branches

AIMS & OBJECTIVES

1. To restore the desired status to the Structural Engineer in construction industry and to create awareness about the profession.
2. To define Boundaries of Responsibilities of Structural Engineer, commensurate with remuneration.
3. To get easy registration with Governments, Corporations and similar organisations all over India, for our members.
4. To reformulate Certification policies adopted by various authorities, to remove anomalies.
5. To convince all Govt. & Semi Govt. bodies for directly engaging Structural Engineer for his services.
6. To disseminate information in various fields of Structural Engineering, to all members.

CONCEPTUAL DESIGN ISSUES IN EARTHQUAKE ENGINEERING

Andrew Charleson

Abstract

In earthquake engineering of building structures, satisfactory seismic performance is dependent upon a sound conceptual design. Even though this process is undertaken at the start of any project, conceptual design must take into account many aspects of design and construction that occur long after the design is complete. The following aspects of conceptual design are discussed; client's expectations and requirements for seismic performance, choice of lateral load resisting systems, levels of design load and ductility, horizontal and vertical regularity, analysis and design, and finally, use of materials and process of construction. These are the primary issues to be dwelt with at a conceptual design phase.

Introduction

This discussion on earthquake engineering and conceptual design issues focus upon the design of new buildings, excluding the topic of retrofitting from its scope. There is a large body of theory and practical experience related to dealing with the earthquake performance of existing buildings, and this is a topic that deserves to be treated in its own merit.

The importance of achieving a sound conceptual design of a building is emphasised by studying the damage buildings suffer during earthquakes. One study reviewed twenty-nine earthquake reconnaissance reports after nineteen earthquakes in developing countries from 1990-1998 (Charleson and Fyfe 2001). After analysing the causes of damage in buildings that had not completely collapsed, it was found that they fell into three categories; conceptual, detailing and construction. Conceptual flaws accounted for 44% of the damage in residential construction and 52% in non-residential buildings. Detailing inadequacies accounted for 30% of the damage, while poor construction accounted for the remainder. The major examples of poor conceptual design were soft-storeys, short columns and irregularity of plan stiffness. Eight other causes of damage seem to be related to inadequate seismic concepts included; vertical discontinuity of infill walls, slender walls, pounding, one-directional structural systems, lack of redundancy, high roof mass, large diaphragm openings and inadequate roof bracing. The percentages of damage attributed to poor design concepts are likely to be higher than what were reported. For example, a poor structural layout or configuration increases structural demands on both detailing and construction quality. Depending on the intensity of earthquake shaking these inadequacies may be exposed. However, if a design concept is sound, neither detailing nor construction may be severely tested.

Another reason for emphasising the importance of the conceptual design phase in earthquake engineering results from the, unusual if not unique, approach applicable to seismic design involving what could be termed 'under-design'. When designing for gravity loads, structural engineers ensure that structural members are stronger than the factored ultimate limit state loads to be resisted. However in seismic design, structural members are designed, using code-sanctioned approaches for a relatively small percent of the inertia loads acting during the design earthquake. Some structures are designed for as little as 10 – 20% of the expected inertial forces. This under-design approach is justified on the basis of the reversed dynamic load cycles during an earthquake, and more particularly, on the expectation of the actual provision of structural ductility. It is expected that these structures spend most of the duration of the design earthquake in the inelastic state. They consequently suffer significant structural damage but that does not lead to collapse due to sound structural configuration and high quality structural detailing, or in other words, due to the application of Capacity Design principles (Park and Paulay 1975).

The dynamic behaviour of a building, much of it in the inelastic range, is very complex. There are so many complicated aspects of seismic performance that most designers are either unaware of or can't simulate in their computer modelling. Codes of practice attempt to account for these complexities in behaviour, and through enormous research efforts over the years they require designers to follow certain rules to ensure reasonable behaviour.

A sound design concept will therefore acknowledge the complexities and uncertainties unique to earthquake engineering. It will do this by trying as much as possible to introduce structural simplicity. That means, keeping force paths as direct and as uncomplicated as possible, and identifying and ensuring dependable ductile mechanisms.

Design concept issues

Having established the crucial importance of conceptual design in the process of achieving a structure that performs well seismically, the following six key issues will be discussed:

- Client expectations and requirements for seismic performance
- Choice of lateral load resisting systems
- Levels of design load and ductility
- Horizontal and vertical regularity of structure
- Analysis and design
- Materials and construction.

Client expectations and requirements for seismic performance

Any seismic conceptual design should be preceded by a conversation with the client. The client must be questioned about his or her requirements regarding the seismic performance of the built facility. For the sake of the client, seismic design issues need explicit and collaborative attention. It is not morally or professionally enough for an engineer to merely satisfy the legal obligations set by city authorities and comply with standard building code requirements. Clients need to be informed of the basis of a seismic design and the expected seismic performance of the building. Any unrealistic client expectations, such as the building remaining undamaged in high intensity shaking, need to be discussed and probably dispelled.

Ideally, very early on in the project, shortly after the architect begins work, the structural engineer should be present at a meeting with the client to discuss seismic design options. Some deep issues are likely to be raised. Most clients are surprised to hear that their new ductile building may be designed for as little as 10% of the actual seismic forces expected during a design-level earthquake even though it complies with current structural codes. The engineer needs to advise them of the probability of the design-level earthquake occurring during the life of the building and the extent of damage likely to be sustained. This discussion will also raise questions like; 'Will the damage be repairable and at what cost?' and; "How long might the building be out of operation following a quake?" The seismic performance of non-structural components also needs exploration at this stage of the design.

Table 1 facilitates discussion of seismic standards and building performance (FEMA 389 2004). Identical tables for the earthquake performance of non-structural components and the continuation of building function: structural and non-structural should also be filled out. Before meeting a client the structural engineer should shade in the cells of the table that apply if a conventional level of seismic resistance is provided; namely, application of minimum code design standards. It is also helpful if the engineer, with knowledge of local seismicity and code requirements of the region, suggests Modified Mercalli intensity values and average return periods for each of the three levels of shaking. During discussion a client's expectations and requirements can be compared to those normally met by application of the minimum code standards. Clients can consider improving aspects of the seismic performance of their proposed building in order to manage their seismic risk objectives. Many clients will be content with conventional seismic design practice, but others will welcome the opportunity to increase the seismic resilience of their new facility. This may involve the introduction of new technology such as dampers, seismic isolation or damage-avoidance technology. For some, earthquake insurance or the lack of it, may be among the factors to be considered when deciding upon seismic design standards. Perhaps, rather than paying annual insurance premiums, clients may choose to carry the risk of seismic damage themselves by requesting enhanced seismic performance.

Table 1. A checklist to facilitate discussion of earthquake expectations between architect and client. (Adapted from FEMA 389, 2004)

For a client without first-hand experience of a damaging quake, the architect and structural engineer could provide preparatory reading material before discussions begin (Stewart 2005). If the client has read a realistic earthquake scenario appropriate to the location of the proposed building all parties approach discussions with reasonably similar perceptions of the damaging quake for which the building is designed. There is a reasonable chance that one day the building and its occupants will be inextricably caught up in and affected by a similar quake.

Choice of lateral load resisting systems

There are a huge number of possibilities when it comes to deciding upon seismic lateral load resisting systems. This may seem surprising when there are basically only three systems to choose from; namely, shear walls, moment frames and braced frames. Of course there are always mixed systems, like a combination of shear walls and frames in one direction, but apart from high-rise buildings it is always better to keep to just one system in each orthogonal direction in order to keep force paths simple and to avoid unexpected seismic damage.

However, the choices of systems become fewer after the engineer is aware of architectural requirements. After all, the structure should seamlessly integrate with architectural planning, and at the same time enhance the architectural design concept or idea (Charleson 2005). These issues will affect the how the structure reads in elevation and in plan, and will help decide how many structural members are necessary and what should be the primary structural material. Questions like "Would it be better to have three or four shear walls in a certain direction?" can be answered only after an awareness of architectural expectations. Once that answer is agreed upon and after a check shows that the number of walls provides sufficient strength and stiffness, then attention can focus upon more detailed considerations such as; will the walls be solid or penetrated, rectangular in elevation or stepped or tapered, how long and thick should the walls be, and how thick to design the chords or flanges of the walls to prevent flexural/compression buckling?

Also at this stage of structural development the issue of horizontal deformation becomes important to consider. Before an architect can calculate the exact dimensions of a building, the maximum horizontal seismic deflections must be determined. According to codes of practice, it is unacceptable for any building to sway over its boundary line and pound its neighbour. Buildings, therefore, are built back from the boundaries (except along street frontages) and clear seismic separation gaps are required. If a building is designed to satisfy maximum code flexibility requirements, these gaps become quite wide. In an example of a ten storey building, a designer must provide a 700 mm wide gap (Charleson 2008). If such a wide gap on both sides of a building is unacceptable the structural engineer must design a stiffer structural system. Perhaps ductile frames will be replaced by less ductile frames of considerably greater stiffness, or even frames are substituted by shear walls.

Levels of design load and ductility

As mentioned above, discussion with the client should precede final decisions regarding the design load and ductility. A low design load means the structure will require higher ductility and be subject to more seismic damage in any moderate to large earthquake. Although codes of practice tell designers what the minimum design load should be for a given structural system and ductility assumption, designers should check that minimum standards which equate to maximum damage and widest horizontal seismic separation gaps are in the client's best interest. Another factor to consider is that high-ductility structures require considerably more sophisticated seismic detailing. So, although the structural footprint of high ductile structure will be smaller than that of a low ductile alternative, the cost of construction may be similar. Not only will separation gaps to boundaries be less for a stiffer building but so will the cost of forming and covering separation gaps between structure and non-structural elements, like glazing or rigid partitions.

Horizontal and vertical regularity of structure

As compared to the seismic performance of individual structural members like beams or columns, building configuration implies a holistic view of a building from a seismic perspective. To a considerable extent the quality of a building's configuration determines, more than many other factors, how well it survives strong shaking. Christopher Arnold, who has written extensively about configuration issues states: "While configuration alone is not likely to be the sole cause of building failure, it may be a major contributor. Historically, before the use of steel and reinforced concrete construction, good configuration was one of the major determinants of good seismic performance" (Arnold 1984).

Structural engineers need to take a very careful approach towards horizontal regularity. It is suggested that they adopt the K.I.S.S. principle–keep it simple and symmetrical, preferring floor plans as well as structural layout to be as regular and symmetrical as possible. Code requirements, reinforce these values. As the European seismic code reminds us: 'To the extent possible, structures should have simple and regular forms both in plan and elevation. If necessary this may be realized by subdividing the structure into dynamically independent units.' (Eurocode 2004). In their quest for regularity, engineers approach configuration irregularities with the aim of minimizing or eliminating them.

Codes provide definitions of irregularity with greater or lesser degrees of preciseness. For the purpose of guiding structural engineers how to approach the design of horizontally irregular structures, one code lists and defines five types of horizontal irregularities in order to classify a building either regular or irregular (ASCE 2006):

- Torsional and extreme torsional,
- Re-entrant corner,
- Diaphragm discontinuity,
- Out-of-plane offsets, and
- Non-parallel systems.

Irregularity leads to a far more time-consuming period of structural design. Whereas regular structures may be designed by simple and straightforward methods, irregular structures necessitate far more sophisticated approaches. Usually the structural engineer constructs a complete 3-D computer model of an irregular structure before subjecting it to code-specified seismic forces. And even such a complex analytical process, whose accuracy is limited by uncertainties inherent in modeling assumptions, can not guarantee perfect seismic performance. Based on observations of quake-damaged buildings, experienced engineers acknowledge that the performance of buildings with irregular horizontal configuration is unlikely to be as good as that of more regular structures.

Irregularity leads to other structural disadvantages. Codes may require structural connections and members to be stronger than normal and therefore they become more expensive. Codes may also penalize irregularity by requiring larger design-level forces. One code requires torsionally irregular structures to be designed 50% stronger (Kitagawa and Takino 1994). But the ultimate penalty for irregularity is the withholding of permission to build. This is the case where at least one code prohibits irregular structures being built in regions of highest seismicity (ASCE 2006).

Another important issue to be resolved at the conceptual design stage is the provision of torsional resistance. Post-earthquake reconnaissance reports invariably show examples of partially or fully collapsed buildings caused by torsion. Consequently, designers try to minimize torsional eccentricities – the distances between centres of mass (in plan) and centres of rigidity or strength. As in most of the structural progress made at this early conceptual stage of a building design, interaction with the architect is vital. It is necessary to place as many of the vertical structural elements resisting seismic load as far apart from each other, or as near the perimeter of the building as possible. Then the long lever-arm enables torsion to be resisted in the stiffest possible manner, and without requiring excessive lateral strength.

Buildings that have suffered seismic damage due to re-entrant corners occasionally feature in earthquake reconnaissance reports. Although re-entrant geometries can take many shapes, what they share in common from a seismic design perspective is their potential for damage resulting from the different dynamic properties of each wing of the building.

The attitude of most codes towards re-entrant corners is to require engineers to undertake a 3-D dynamic analysis where the length of a projecting area of building causing a re-entrant corner exceeds approximately 15% of the building plan dimension. It is necessary to design the re-entrant structure to avoid diaphragm tearing and excessive horizontal deflections. This improvement can be achieved by fine-tuning the relative stiffness of the wings. However if they are long or if diaphragms are weakened by penetrations for vertical circulation or other reasons in the critical region where they join, that approach may not be structurally sound. The building might best be separated into two independent structures.

In the ideal world of the engineer, diaphragms in buildings are not penetrated by anything larger than say a 300 mm diameter pipe. Diaphragms should also be planar and level over the whole floor plan. However the real world of architecture is quite different. In most buildings quite large penetrations are required for vertical circulation such as stairways and elevators. Diaphragm penetrations or irregularities need to be identified and strategies to overcome them need resolution at the conceptual design stage.

Vertical irregularities repeatedly observed after earthquakes to have initiated severe damage include:

- a floor significantly heavier than an adjacent floor,
- vertical structure of one storey more flexible and/or weaker than that above it,
- short columns,
- discontinuous and off-set structural walls, and
- an abrupt change of floor plan dimension up the height of a building.

Most of these irregularities, like the horizontal irregularities discussed above, are described and defined in more or less detail by seismic design codes. Each configuration irregularity modifies the dynamic response of a building and increases structural and non-structural damage. Some minor to moderate irregularities are acceptable to engineers and design codes, and are dealt with by applying more sophisticated design techniques. Poor vertical configuration entails force path discontinuities or complexities that affected structural members are generally unable to cope with without severe damage.

Of all vertical configuration problems, the soft storey is the most serious and is by far the most prevalent reason for multi-storey building collapses. So many buildings, located in seismically active regions throughout the world possess relatively open ground floors and are at risk of a soft storey mechanism forming. A report on the 1995 Kobe earthquake observes that ground floor collapse was the most common failure mode in small commercial and mixed-occupancy buildings. Regarding larger commercial and residential buildings which in most cases appeared regular from the street, the report notes: 'Partial or full collapse of a single story of buildings was the common "collapse" failure mode.. .. The particular story that sustained partial or full collapse varied from building to building ....' (Comartin 1995).

Soft storey behaviour is often caused by weak columns and strong beams. Without the Capacity Design approach, columns are usually weaker than beams so columns alone have to sustain damage. Soft storeys are also caused by other configuration irregularities, such as by the combination of open ground floors and masonry infills. A strategy of either separation or differentiation can overcome soft storeys where they are required architecturally. Separation involves isolating stiff and strong elements like infill walls and deep beams that cause adjacent elements like columns to be relatively more flexible and weaker, from the force path. Differentiation describes a design approach that clearly distinguishes between gravity-only and seismic resisting structure and ensures selected members primarily resist either seismic or gravity forces. For example, this approach allows some slender, apparently soft storey columns, only because elsewhere in plan there are strong moment frames or shear walls that are designed to resist all lateral loads.

Short columns need to be eliminated at the conceptual design phase. By definition, short columns possess a very short distance over which they can flex horizontally. The problem is that the free-length is too short to allow for the development of ductile plastic hinges. In the event of seismic overload the column fails in shear. Once opposing diagonal shear cracks form in a reinforced concrete column its reduced gravity-carrying capacity often causes collapse.

The danger posed by off-set walls supported on cantilever beams has been tragically and repeatedly observed during five Turkish earthquakes in the 1990s. After categorizing building damage a report concludes: 'Buildings having architecturally based irregular structural systems were heavily damaged or collapsed during the earthquake. Cantilevers of irregular buildings have again proven to be the primary source/cause of seismic damage. Many buildings have regular structural systems but [even if] roughly designed performed well with minor damage' (Sesigür 2001).

Seismic codes categorize buildings with abrupt setbacks as irregular. Sophisticated structural analyses can quantify the 'notch effect' of a setback, but even though structural engineers avoid notches wherever possible because of stress concentrations, setbacks can be designed satisfactorily.

Analysis and design

During the conceptual design process the lead structural engineer needs to be aware of how the detailed structural analysis and design will proceed. The structure has to be modelled and then have loads applied to it. While there is plenty of suitable software available it is more of a challenge to achieve a computer model that represents faithfully the proposed real structure. Having completed an analysis the design continues, and now the software output needs to be handled with care. Usually elastic analyses are undertaken, but as we are aware, much of the life of a structure during an earthquake occurs in the inelastic state. So, design consists of a lot more than taking elastic results and then designing structural cross-sections. To achieve the level of ductility assumed and required, hierarchies of strength within the structure must be achieved by using the Capacity Design process. Most computer output will require factoring to achieve a suitable and reliable post-elastic performance.

Materials and construction

The final conceptual design issue to be discussed at the beginning of a design relates to materials and construction, even though construction occurs long after design has been completed.

Because of the reliance upon ductility as the most common strategy for buildings surviving a damaging earthquake, the materials of construction, at least in areas where plastic hinges and fuses will form, must be ductile. In reinforced concrete construction this means using ductile reinforcing steel in those special regions expected to be subject to damage. If non-ductile steel is used then two problems arise. First, the non-ductile steel may not cope with the extent of inelastic tension and compression strain, and break. Secondly, if the steel yields and strain hardens, its strength might be significantly higher than expected. This could mean that the member over-strength causes premature and undesirable yielding in other members. For example, in the strong column-weak beam philosophy the beams of moment frames must be the weaker elements. If they force yielding in columns, then severe overall building damage can occur.

While reinforcing steel in areas of structural members that have been selected to exhibit structural ductility must be ductile, in other areas of the building that are designed not to yield, a less ductile steel may be used. But even then designers must check that steel can accept bending during fabrication without fracture or be welded without losing strength. This is why reinforcing steel should be of a specified quality.

If ductile steel is unavailable, then at the conceptual stage of design a non-ductile design approach should be taken. The same types of consideration apply to the strength of concrete. If a region has a reputation for producing low strength concrete even when high strength is specified, high performance ductile structural systems like ductile moment frames should be avoided.

Steel and concrete properties raise the issue of quality assurance. Without a robust quality assurance programme the designer can have little or no confidence that the material properties specified will be achieved in the field and that construction will be in accordance with the working drawings. Clients are the ones most affected by a lack of quality. A building that was expected to perform well during an earthquake, due to a lack of material and construction quality, might collapse. This problem is so often reported in earthquake reconnaissance reports it can't be ignored. Once again, if quality assurance is likely to be an issue, then designers should use the least sophisticated structural system of all to resist seismic forces - shear walls.

Buildings of smaller scale

In the previous sections of this paper the assumption has been made that the buildings being designed receive architectural input and are higher than non-engineered construction would allow. So what about conceptual design issues for smaller, perhaps more humble buildings? Even these need a clear design strategy to achieve satisfactory seismic performance. Take adobe houses as an example. A sound design concept will include provision of tie members and details, at least at eaves level, to prevent walls from separating from each other and overturning. Also the numbers and sizes of openings need to be limited so there are sufficiently long shear panels in each direction to resist seismic forces. There are other factors to be considered as well, but these have been well documented by others.

Summary

The paper has argued that satisfactory seismic performance is dependent upon a sound earthquake engineering conceptual design. The following aspects of conceptual design were discussed; client expectations and requirements for seismic performance, choice of lateral load resisting systems, levels of design load and ductility, horizontal and vertical regularity, analysis and design, and finally, materials and construction. All of these subjects must be attended to at a conceptual design phase.

A. Earthquake performance of structure

	Damage			
Intensity of shaking	Severe to non-repairable	Repairable: evacuation	Repairable: no evacuation	No significant damage
Low				
Moderate				
High				

B. Earthquake performance of non-structural components

	Damage			
Intensity of shaking	Severe to non-repairable	Repairable: evacuation	Repairable: no evacuation	No significant damage
Low				
Moderate				
High				

C. Continuation of building function: structural and non-structural

	Damage			
Intensity of shaking	Severe to non-repairable	Repairable: evacuation	Repairable no evacuation	No significant damage
Low				
Moderate				
High				

References

- (2004). Eurocode 8: design of structures for earthquake resistance – Part 1: general rules, seismic actions and rules for buildings (BS EN 1998–1:2004). European Committee for Standardization.
- (2006). Minimum design forces for buildings and other structures (ASCE/SEI 7-05). American Institute of Civil Engineers.
- Arnold, C. (1984). Building configuration: the architecture of seismic design. Bulletin of the New Zealand National Society for Earthquake Engineering, 17:2, 83-8.
- Charleson, A. W., 2008. Seismic design for architects: outwitting the quake. Elsevier, Oxford, UK.
- Charleson, A. W., 2005. Structure as architecture: a source book for architects and structural engineers. Elsevier, Oxford, UK.
- Charleson, A. and Fyfe, G.D, 2001. Earthquake Building

Damage in Developing Countries: a Review of Recent Reconnaissance Reports, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 34, No. 2, pp.158-163.

- Comartin, C. D., Greene, M. and Tubbesing, S. K. (eds.) (1995). The Hyogo-Ken Nanbu Earthquake: Great Hanshin Earthquake Disaster January 17, 1995, preliminary reconnaissance report. Earthquake Engineering Research Institute.
- FEMA 389, 2004. The table is based upon a similar table in Applied Technology Council et al (2004). Primer for design professionals: communicating with owners and managers of new buildings on earthquake risk (FEMA 389). Federal Emergency Management Agency, U.S.
- Kitagawa, Y. and Takino, F. Chapter 24 Japan. In Paz, M. (1994). International handbook on earthquake engineering: codes, programs, and examples. Chapman & Hall.
- Park, R. and Paulay, T., 1975. Reinforced concrete structures. John Wiley & Sons.
- Sesigür, H. et al (2001). Effect of structural irregularities and short columns on the seismic response of buildings during the last Turkey earthquakes. In Corz, A. and Brebbia, C. A. (2001). Earthquake resistant structures III. Proceedings of the International Conference of Earthquake Resistant Engineering Structures (3^{rd}). WIT, 83-90.
- Stewart, M. (ed.) (2005). Scenario for a Magnitude 6.7 earthquake on the Seattle fault. Earthquake Engineering Research Institute (EERI) and the Washington Military Department, Emergency Management Division.

Author :

Andrew Charleson is an Associate Professor at the School of Architecture, Victoria University of Wellington, New Zealand. He had written books on "Structure as Architecture: a Source Book for Architects and Structural Engineers" & "Seismic Design for Architects: Outwitting the Quake"
Email : Andrew.Charleson@vuw.ac.nz

REPORT : ISSE LECTURE ON "FM GLOBAL'S APPROACH TO SAFE DESIGN PRACTICE TO LIMIT PROPERTY LOSSES" – 11^{TH} MAY 2011

Indian Society of Structural Engineers (ISSE) in association with The Institution of Engineers India (IEI) arranged Lecture on ***"FM Global's Approach to Safe Design Practice to Limit Property Losses"*** On 11^{th} May 2011 at the Auditorium of Institution of Engineers Maharashtra State Centre, Mahalaxmi, Mumbai.

Mr. S G Dharmadhikari, President ISSE welcomed all the delegates on behalf of ISSE and IEI.

Mr. Hemant Vadalkar talked about the outline of the lecture and importance of quality and test procedures required by the global clients. He emphasized that there is no substitute to the quality and better engineering practices if we have to compete globally.

Mr. Robert Azimi head of Global Operation, N. America of M/s. FM Approval talked on ***"FM Global's Approach to Safe Design Practice to Limit Property Losses"***

Safety in Building Design and Limiting property losses is the main consideration for Insurability of buildings. Every structural designer and Architect in the country is faced with challenges of meeting globally acceptable norms for building envelope design demanded by multinational clients directly or by clients servicing end use of multinational companies.

Product presentation was made by sponsors Mr. N. Srinivas of Lloyd Insulations (India) Limited, the leading Design + Build agency for Pre-engineered Buildings who is specialist in FM Approved roofing and Wall Cladding Systems.

The response for the Lecture was overwhelming. It was attended by about 100 delegates.

NAD-AL-SHEBA RACE COURSE DEVELOPMENT PROJECT : CONSTRUCTION OF THREE BRIDGES

Vivek G. Abhyankar

Abstract :

A detailed technical report on the construction of three flyover bridges as a part of Nad-al-Sheba (horse and camel) Race Course development, Dubai, constructed by M/s *AFCONS Infrastructure limited for Roads and Transport Authority (RTA) formerly known as Dubai Municipal Corporation*. Various problems faced during the construction and creative solutions devised by the project team are discussed in this paper. In spite of these, the project was completed in record time, with appreciation of the client.

Key Words :

Continuous construction, Bore collapse, Kentledge method, RFI, Time-Extension. Cable extension, Fixed pier, Monolithic pier, Bridge aesthetics, Shop Drawings, RFI

General :

About fifty years back Dubai was a land of fishermen. Later (about 35 years back) formation of Emirates took place. Seven big cities (now known as Emirates) were connected to form United Arab Emirates (UAE). These seven emirates are namely Abhu-Dhabhi, Dubai, Sharjah, Ajman, Fujairah, Umm Al Quwain and Ras Al Khaimah. Amongst these Abu-Dhabi and Dubai are most favored emirates for the foreigners.

Dubai is know for it's modern structures, rapidly growing civilization, foreign-tourism and trade. It is one of the seven emirates of the UAE where maximum investment is taking place from the people from all over the world. The ambitious project like Burj-Arab tower (Burj-Khallifa), Burj-Dubai, palm-island, world-island and many more upcoming projects have challenged entire engineering community and especially to civil & structural engineering. The Government is continuously putting the challenge to the engineers to modernize the 'habitat' that is Dubai.

Dubai being a dessert area, camels and horses were extensively used by the local Arab people. The climate in this part of the world is very dry, temperature varies from 5° C in winter to 50° C in summer. In summer temperatuer drops during night. Desert storms are commonly seen. In recent years (about last ten / fifteen years, after the Gulf-war), rains have started taking place in every winter. Due to such extremities in climatic conditions, local villagers used to have, horse and camel race as an entertainment, after busy working seasons. Now these races have become a part of their tradition. Dubai city has a large area reserved for race course developments. The area is called as Nad-Al-Sheba. When Dubai government decided to develop this area to international standards. Therefor the whole idea was conceived on International standards, by various experts from all over the world. Team experts, Architects, Planners, Engineers, Finance advisors and authorities were invited to prepare the proposal and then International tender was floated in market.

Afcons being one of the pioneer construction companies in marine structures in India, and getting diversified in other sectors of constructions like Infrastructure, Nuclear and Industrial etc., company was keen to participate in this bid. Afcons already had considerable experience in International construction works, having worked at locations such as – Mauritius, Madagascar (Africa), Ruwais (Abhu-Dabhi), Iraq etc. The company decided to participate in the tendering process.

As the tender was on International level, construction companies from all over the world participated in it. But final RTA shortlisted Afcons, as the technical and financial bid was most competitive. Though it was not a new experience for the company to work on foreign lands, the responsibility to fulfill the expectations of RTA was a challenge before the project team.

Road and Transport Authority (RTA), formerly known as Dubai Municipal Corporation had stringent norms of construction. The entire project was monitored by experts from various parts of the world under M/s Africon and Al-Burj. Projects under RTA are not given 'Time' and 'Cost' extension. The contractor delaying on projects may even get black-listed, and are not allowed to perform on any works in future. Officers from Britain, Switzerland, Sudan, Pakistan, Iraq, Sri Lanka were appointed by the company to execute the work. The client also had a team of dedicated and competent engineers.

The overall project of 'Nad-al-Sheba' race course developments had four parts namely construction of roads and bridges, construction of water canal from sea till the race course (for navigation of visitors from other emirates), construction of main structures / stadium, development of adjoining areas. Amongst these, the first package of construction of roads and bridges was executed by Afcons. The overall work gave lots of experience to the organization and to its entire site team.

Briefs about the Project :

Nad-al-sheba race course is located south of world famous Al-Burj-tower. Sea shore is very close from the race course. Areas like Camel market, falcon house are very close from the project site.

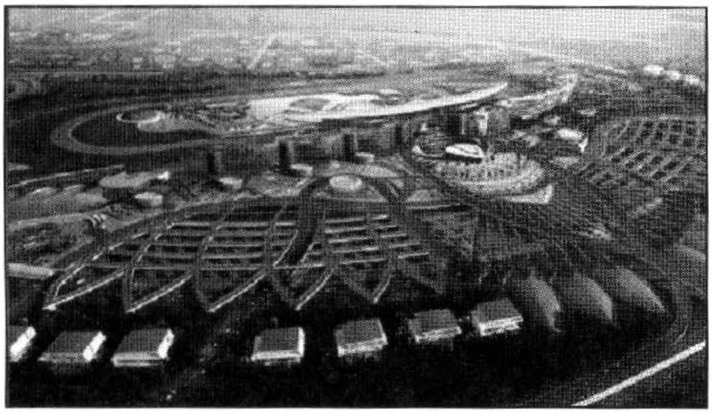


Fig 1. Birds eye view of Proposed look of Nad-Al-Sheba Race course.

The scope of work for Afcons covered construction of three approach bridges (namely MDN-1A, MND-3D and the main bridge) to the race course and construction of main roads within the entree area (Fig-14, shows entire alignment & scope of work).

The fourth bridge called 'VIP' bridge (Fig - 2) was a separate package in this tender. Tendering of VIP Bridge was done after processing first package. It was constructed by M/s Balfour Beatty. Another package was to construct the approach water canal. Construction of canal was done by M/s Dutco. Construction of Stadium building was executed by M/s Mydan. The roads and bridges within scope of Afcons, Structural Design was provided by M/s Al-Burg and Project management consultant was Ms/ Africon (it was JV of Africon & Al-burg). VIP bridge design was provided by a team consisting of M/s DNEC, with Africon UAE, Dutco Balfour Beatty LLC, Petrofab Intl. FZC.

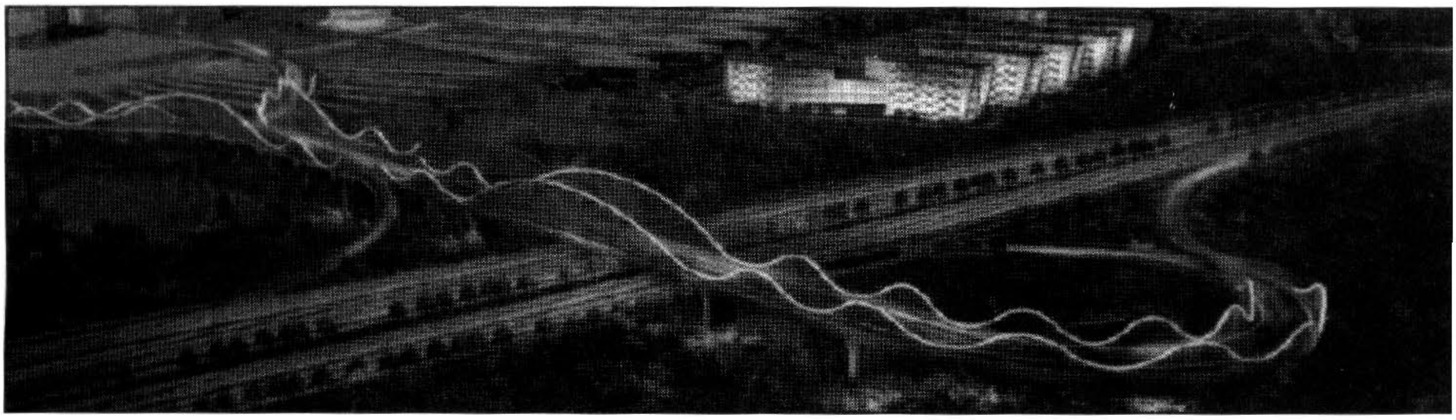


Fig 2. VIP bridge (Constructed by Ms/ Dutco Balfour Beatty LLC)

In the paper, works executed by only Afcons are described. The key detail of all three Bridges are as described below :-

Project Details :

Scope of Work at the Time of start of construction

	Mdn-1a	Mdn-3d	Main Bridge, paths				Total
			North	South	Link-1A	Link-1B	
Total Length (m)	326	390	692	692	148	459	2707
Number of Spans	8	10	17	17	4	11	67
Typical span length (m)	42	42	60	60	36	36	276
Number of Piles	61	78	230	241	40	106	756
Piers, Abutments	7	9	61	62	3	10	152
Bearings	16	20	77	82	8	24	227
R.E. wall Length (m)	380	370	600	600	90	150	2190
Expansion Joints	2	3	4	4	2	4	19

Work executed Re-engineered by Client

	Mdn-1a	Mdn-3d	Main Bridge, paths				Total
			North	South	Link-1A	Link-1B	
Total Length (m)	326	200	440	-	-	-	966
Number of Spans	8	5	11	-	-	-	24
Typical span length (m)	42	42	60	-	-	-	144
Number of Piles	61	42	263	241	22	54	683
Piers, Abutments	7	5	40	28	-	-	80
Bearings	16	10	77	-	-	-	103
R.E. wall Length (m)	380	370	600	-	-	-	1350
Expansion Joints	2	3	4	-	-	-	9

Link-1A, Link-1B were because of financial crisis.
Later the South wing also reduced partially.

Concrete grade used:
- Super structure – M40
- Substructure – M40

Reinforcement Grade used: Fe500

Project Duration : 19 months
Project cost : 498 M Dirham. (after Revision of scope)

Vendors / Suppliers / Sub-contactors :

- Pile-driving – M/s Swiss Boring
- Formwork –
  - Superstructure (M/s RMD Kwikform),
  - Piers (M/s Anvem Steel)
- Reinforcement cutting bending, BBS – M/s Ulmost
- Prestressing system – M/s Nasa Structures
- Bearings, Expansion Joints – M/s Alga bearings and Exp. Jts
- RE walls and Geotextiles – M/s Freyssinet
- Concrete and other construction Chemicals : M/s BASF

Fig-3a – Pile Driving in progress

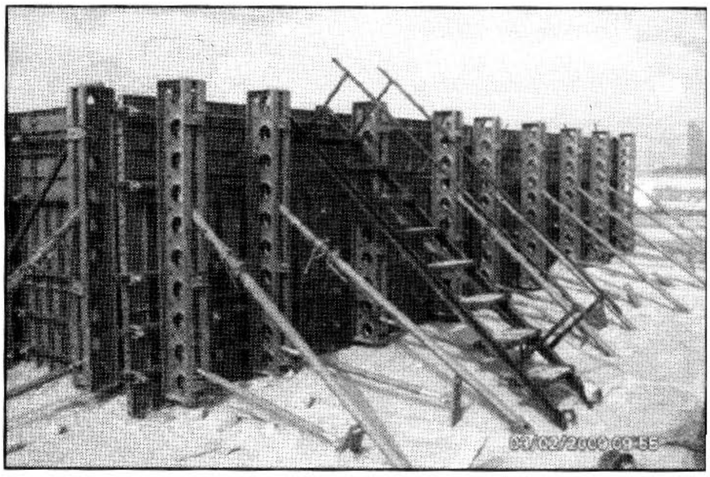


Fig-3b – Pile-cap formwork (M/s RMD Kwikform type)

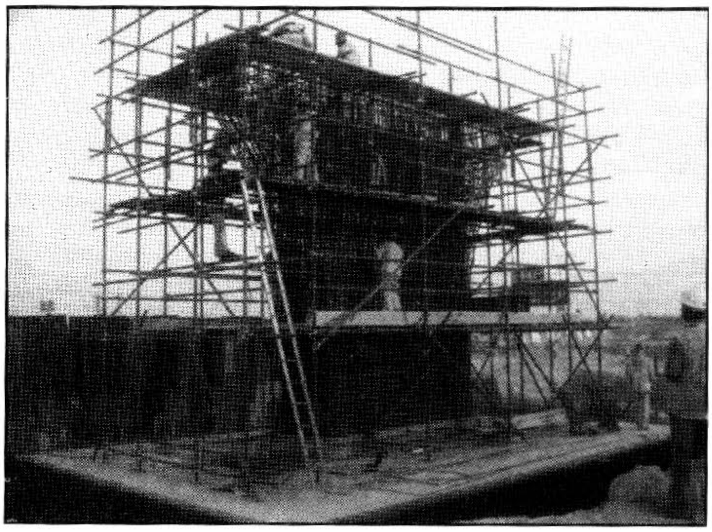


Fig-3c. Reinforcements of the Piers

Piles were driven using Casagrande pile driving rigs. Freshly driven bores were stabilized using 'polymer solution' instead of conventional bentonite slurry. This helped in maintaining the steep profile. A steel liner was used to stabilize top 10m of the freshly driven bore. Designed capacity of each pile below pier cap was 450MT and 600MT below the abutments. As the welding of rebars was not allowed couplers were used for reinforcements in the piles.

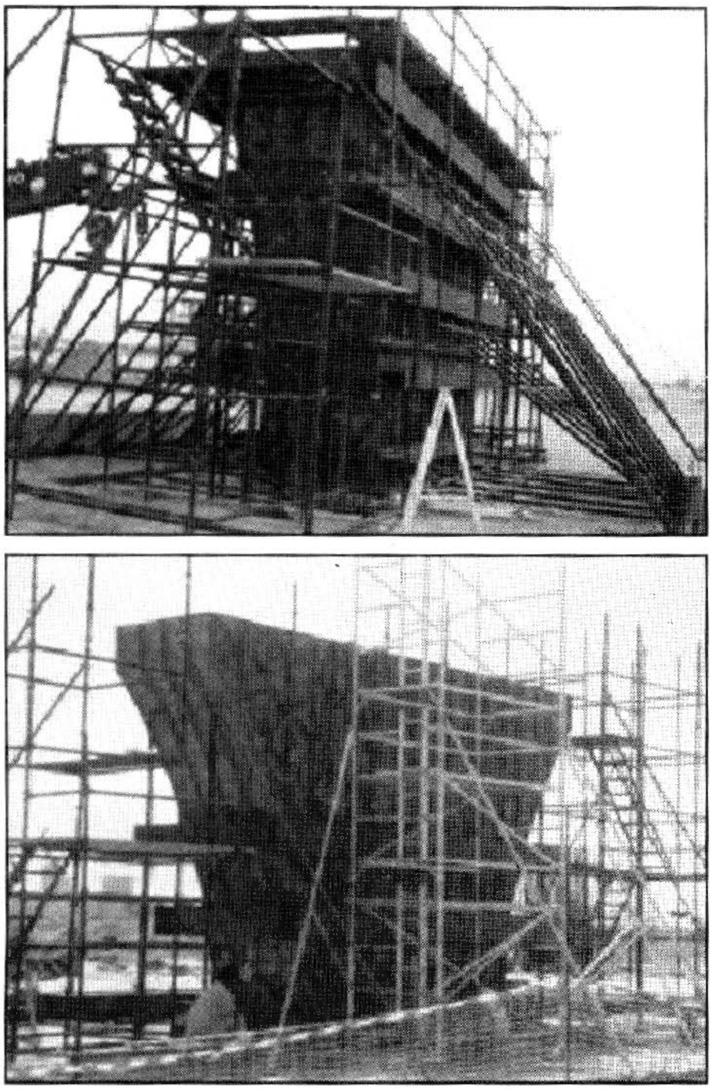


Fig-4. Typical View of Pier Formwork supplied by M/s Anvem

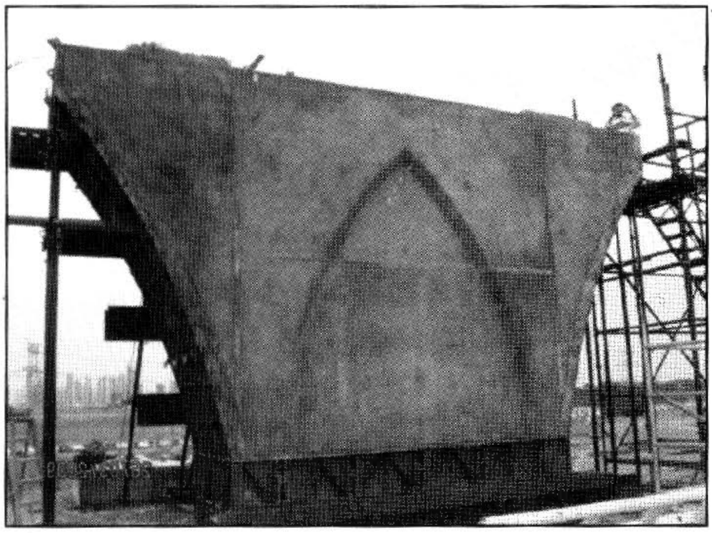


Fig-5. View of Pier after de-shuttering

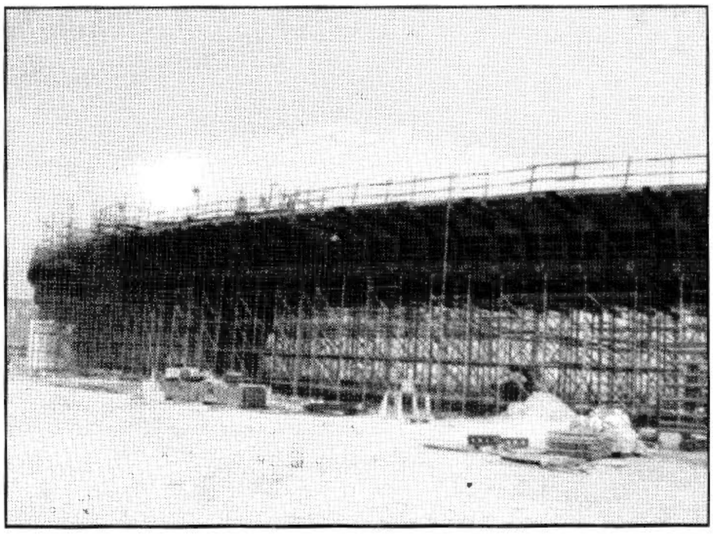


Fig-6. RMDK formwork / staging for the bridge deck

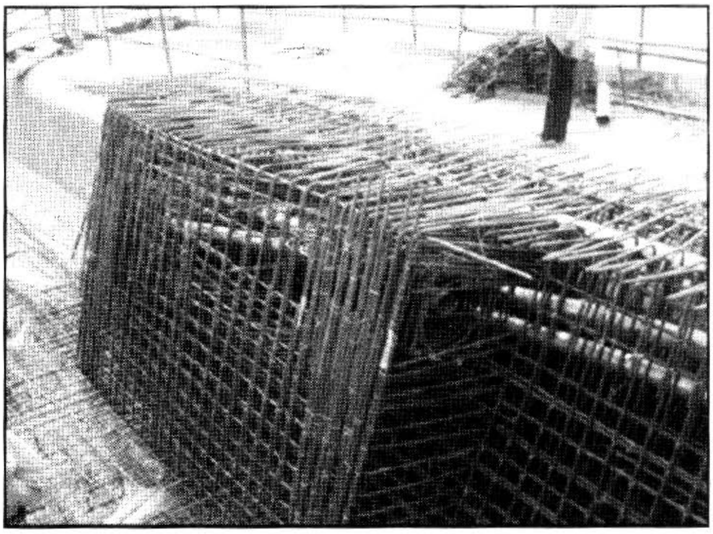


Fig-7. Densely reinforced Prestressed concrete section

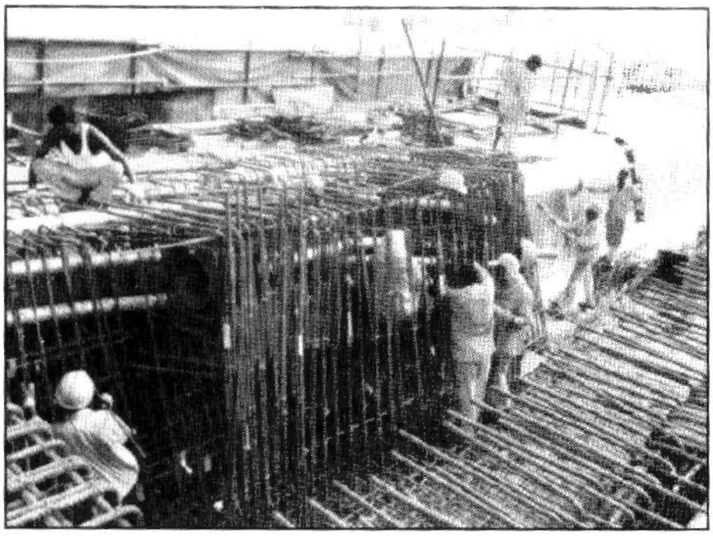


Fig-8. Workers engaged in placing reinforcements and cable ducts at piace

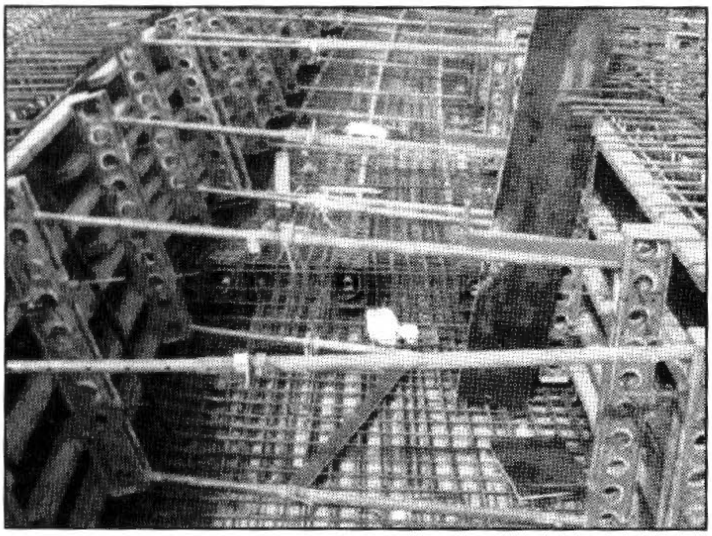


Fig-9. Typical View of Inner Bridge Formwork (M/s RMD Kwikform type)

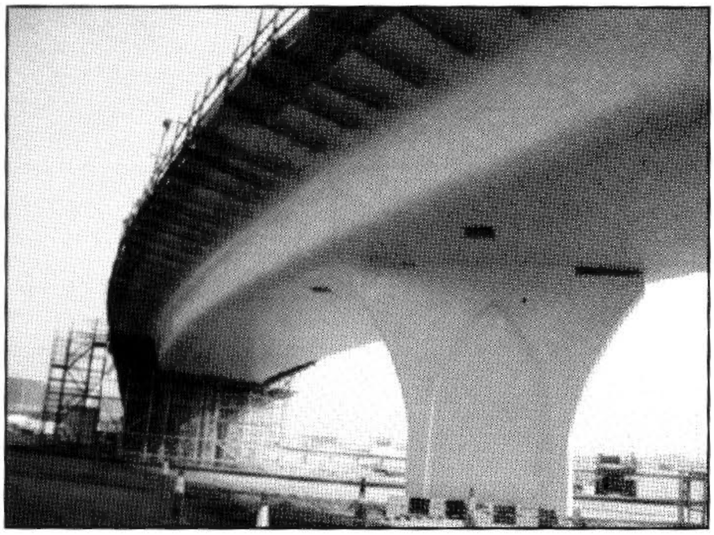


Fig-10. MDN-1A on verge of completion. The white epoxy coating can be easily seen.

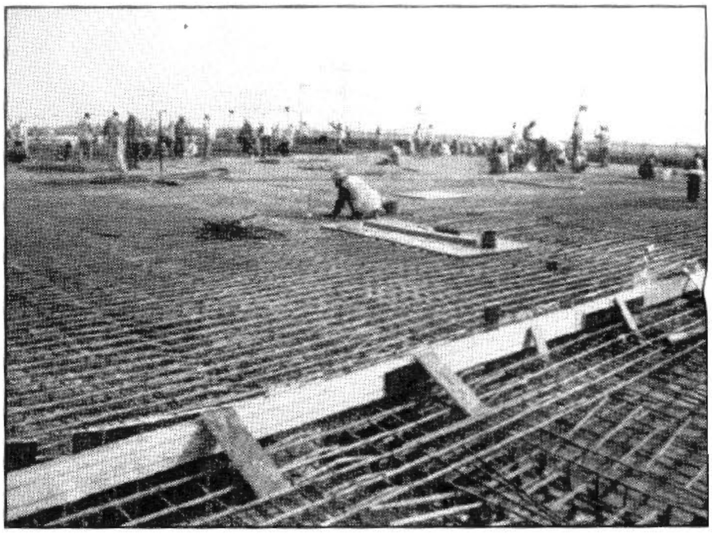


Fig-11. Large width deck slab (<60m) of Main bridge

Fig-12. Due to heat in summer, night concreting of the Large deck was preferable

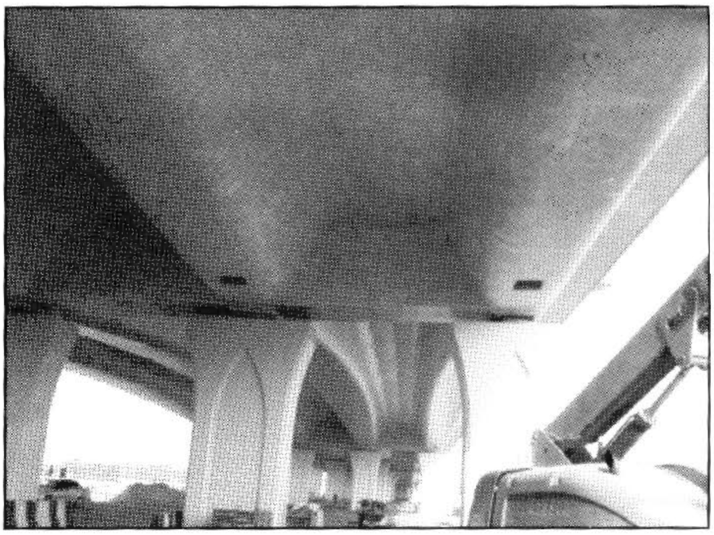


Fig-13. Main Bridge deck (Before applying epoxy coating for high quality of concrete finish.)

Construction Hurdles :

When Afcons acquired the work order of this work in August'2010, immediately our organization started initial mobilization of resources (man, material and machinery), considering importance of the work. Here its worth mentioning that all major works associated with Infrastructural growth in UAE are directly monitored by the Ministry. Soon after acquiring the site, the site team noticed that the clouds of recession / financial crises started shading all the trades in Dubai, including construction projects. The team noticed that most of the mega-projects were on the verge of getting 'Hold' tag. The big projects like 'Lagoons', other real estate projects' etc. were already closed. This created an unstable feeling within the project team members.

After two months of initial mobilization (Oct-Nov'2009), the Project team started working at site. In UAE, Project offices usually open at 6:30am and close down work around 4:00pm, due to extreme heat. This was a rainy season in this part of the world. Rains imposed constraints in road construction / underground works. The time was utilized by the technical team members, to minutely study Construction drawings, prepare action plans, study the methods followed in near by projects by other International construction companies etc. During the study, the team noticed various information missing / errors at a few places. The same was discussed with PMC, and raised the query in the form of RFI. The clients soon had many RFIs. Hence consultant had to appoint dedicated engineers to sort all the RFI. Considering the fast track nature of the project, Afcon's team raided query / request to client (RTA) to pay attention to this. RTA instructed the contractor and consultant both to solve the matter at the earliest without affecting the time and cost estimate. Soon the consultant informed the contractor that there are certain changes expected in the engineering details, under re-engineering for which revised.

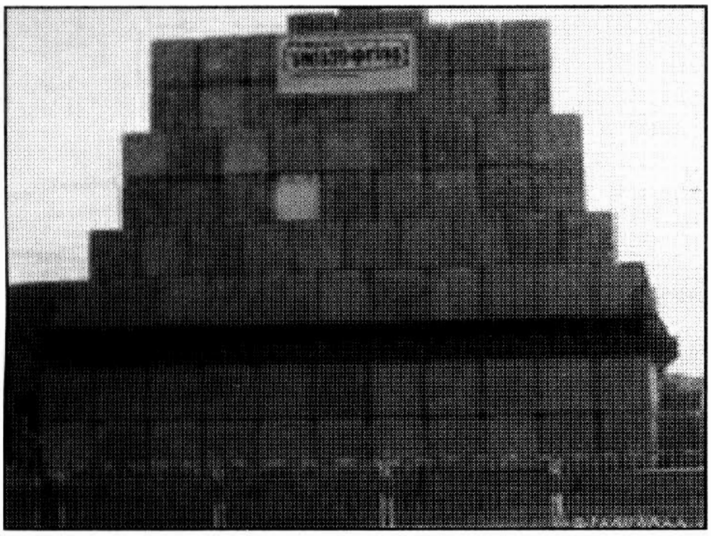


Fig-13a – Pile Load test – kentledge, (down)

Fig-13b Jacks and Load cells on Pile head

In Dubai, it is mandatory for any party executing works to have approval of Dubai municipal Corp. To avoid any delays in approval, our organization decided to appoint an approved agency to start piling works. Various companies' proposals were scrutinized and M/s Swiss Bore was appointed to start the work. As per original contract, bore-cast in situ pile of 1000mm diameter and 32m depth were constructed. After constructing the first pile, the same was load tested (using kentledge method). The 32m pile showed very high capacity during the test. Hence PMC and client asked to perform one more test on 24m deep pile (the 24m depth was estimated by interpolation, though it was not a exact approach, but fairly logical for friction piles in uniform strata like that in Dubai).

The 24m pile was then driven and tested which also passed the required load and settlement criteria. The site team proceeded ahead with iling of bridge MDN-1A with 24m depth. PMC then started pressurizing the contractor to further reduce the length of the pile to 18m. The site team rechecked the same with the tender provisions and soil investigation data provided by the client. Finally 18m pile was driven and tested at second bridge MDN-3D. The same passed the requirements. The consultant asked the site team to proceed with the 18m length of the pile for remaining work at MDN-1A and all piles of MDN-3D. This iteration consumed lot of time.

When piling work of bridge MDN-1A was about to get over, and piling work of MDN-3D was just started the PMC asked contactor to test a 16m deep pile and test the same at last bridge. The was done, and also passed the load-settlement requirements. The client and PMC both were very happy on this cost saving. The PMC wanted to now check the 12m deep pile. Here the Structural Consultant of the bridge had to intervene, asking to check the minimum pile length to resist the lateral loads, yet PMC got the approval from client to get the 12m pile checked.

Dubai's soil stratum is a typical example of fine sandy strata with weak empty soil-pockets formed during dust-storms. The same was experienced while driving 12m deep pile, where the bore suddenly collapsed inwards. Because of this, clients instructed to stop further eengineering and proceeded ahead with the 16m deep piles. Unfortunately, tendered quantity become almost half. Though the PMC and the consultants, agreed with 16m depth of the pile, the second step of re-engineering was started to reduce the number of piles itself. But as the final pile of 16m was just meeting the requirements, the number could not be reduced drastically.

The third step of re-engineering was tried on the reduction of bridge length in MDN-3D. Such reduction was required because during the planning stage the minimum ground clearance for the bearings was not followed by the Planner / Consultants. Thus the initial few months time (four to five moths after initial mobilization period of two months) was critical. The organization could appoint a Project Manager from Dubai to speak local language (Arabic), and could communicate with the clients and PMC in a better way, to keep eye on the project time loss. Then the organization decided to work on 24x7 basis work on three shifts in full swing were started.

All these delays, trials, iteration in first five / six months gave lots of experince, which was used to trouble shoot all the further hurdles / issues. The challenge was to maintain quality, without losing time, money and yet to give safe construction. Now the organization geared-up to do the work on time. Many other problems as listed below were faced by the team, but all were carefully addressed with sound engineering solution, to meet the target :-

1) Delayed payments
2) Alteration / reduction in scope of work under re-engineering
3) Delay in acceptance to RFI by PMC, action taken on RFI
4) Construction workers' issues
5) Idling of construction resources (machine / men) in absence of approval / drawings
6) Administrative issues (pressure from other agencies / contactors on client)
7) Relocation of utilities
8) High magnitude of pre-stressing
9) Incompatible values of stressing mentioned in drawings, especially at cable coupler locations

Outcome of this project :

The initial period of a few months was difficult for the contactor. Both the parties were new to each other. But once they understood the working style and the project requirements in Dubai / RTA (client), the team members could jointly overcome all the hurdles. Finally the finished quality of output (as seen in various photographs) was obtained. The credit also goes to the various vendors, supplier and the subcontractors, as each one did their job 'the best'.

Conclusion :

Very limited number of contactors from India take a challenge to participate and construct international projects. But such contacts give lots of experience, understanding about International construction practices, legal and contractual provisions. Yet, there is scope for significant financial gains. Also such projects help organizations to earn lot of respect in

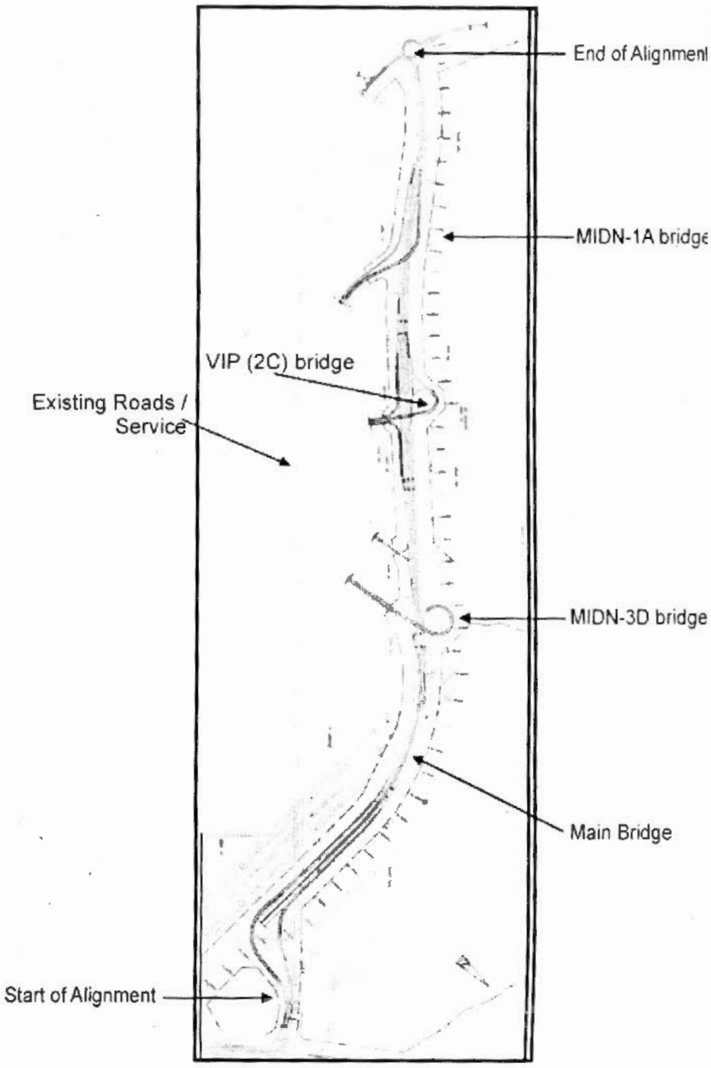


Fig 14. Overall scope of the work (showing road alignment and the Three Brides)

the International construction industry. It's worth mentioning here that after completion of this project Afcons name was put in the list of a few preferred contactors to do job in Dubai.

Acknowledgement:

The author is thankful to the Afcons Infrastructure Limited and entire site team for making all the relevant data available for this paper.

Author:

Shri. Vivek G. Abhyankar – C.ENG (Sr. Manager (Design) – AFCONS Infrastructure limited.
E-mail : abhy_vivek@yahoo.com

SERVICES TAX FOR CONSULTING ENGINEERS

ISSE had written a letter to finance ministry requesting them to charge services tax on the amount received by consulting engineer and not on the basis of bill raised by engineers. Ministry acknowledged the letter from ISSE. The copy of letter is enclosed for the reference all members.

Dy. No. 93044

सत्यमेव जयते

SURESH YADAV

निदेशक
वित्त मंत्री कार्यालय
नई दिल्ली-110001
भारत

DIRECTOR
FINANCE MINISTER OFFICE
NEW DELHI-110001
INDIA

16 JUN 2011

Dear Shri Dharmadhikari,

This is to acknowledge receipt of your letter dated 25th May, 2011 addressed to Hon'ble Finance Minister Shri Pranab Mukherjee, on Point of Taxation Rules 2011 – Service Tax.

Your letter has been forwarded to the concerned officer for taking necessary action.

With regards,

Yours sincerely,

(Suresh Yadav)

Shri S.G. Dharmadhikari
President
Indian Society of Structural Engineers
C/o S.G. Dharmadhikari
24, Pandhi Niwas, S.K. Bole Marg
Dadar (W)
MUMBAI – 400 028

Important News

Efforts of ISSE in representing to finance ministry has resulted favourably. Now, Consulting Engineers can pay Service Tax on Receipt basis for Proprietor, Individuals & Partnership firms, as done earlier.

APPEAL TO ISSE MEMBERS

1) STRUCTURAL AUDIT WORK

ISSE wants to from a panel of structural engineers for carrying out structural audit of the buildings. Those who are interested in carrying out structural audit work may please contact ISSE office.

2) PROOF CHECKING OF STRUCTURAL DESIGNING WORK

ISSE wants to from a panel of structural engineers for carrying out proof checking of structural designing work. Those who are interested in carrying out proof checking of structural designing work may please contact ISSE office.

Tel (022) 24365240 / 24221015 / 24314445 Email – issemumbai@gmail.com

3) ISSE Guidelines for minimum fees to be charged by structural consultant

ISSE would like to publish fees structure for consultants. Members are requested to forward their views / suggestion on minimum fees to be charged by structural engineers.

4) National Disaster Management Authority (NDMA)

National Disaster Management Authority (NDMA) had published guidelines for better structures. RBI had issued a circular to banks for sanctioning loans on NDMA guidelines. For more details refer a link www.ndma.gov.in

UNDERSTANDING OF STRESS FLOW PATTERN USING STM & FEM

Prasad Samant, Hemant Vadalkar

Introduction:

While analyzing the structure, it is very essential to visualize stress flow pattern. It is even more essential in complicated structure such as deep beam (girder), pile cap, corbels, dapped-end beam. For stress flow analysis purpose, two methods are used namely strut and tie method (STM) and finite element method (FEM). In STM, truss members are loaded with uni-axial stress that is parallel to the axis of the stress path. FEM analysis is done by using computer software STAADPro.

Example considered for Analysis:

a) A girder supporting floating column (carrying axial compression)

Simply supported girder is considered having 6M span with depth of 2M and width of 500MM carries two concentrated factored load of 1600KN each 2M from respective support.

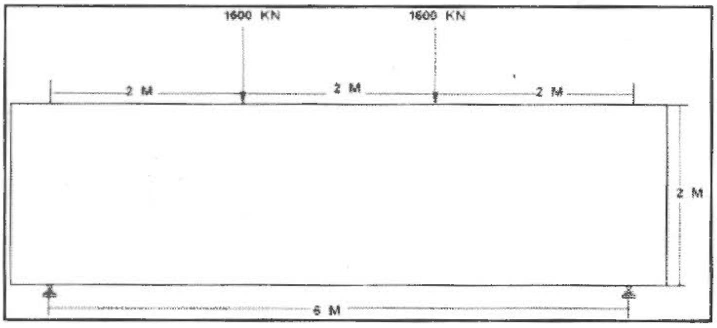


Fig.1

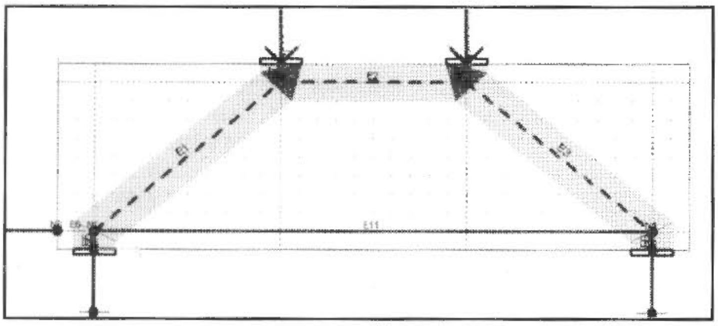


Fig.2

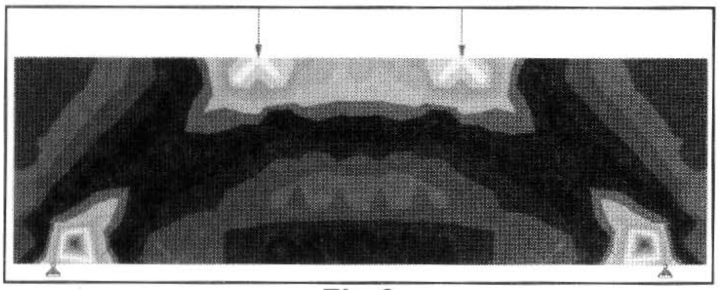


Fig.3

Load from floating column at top of girder flows to the support as shown in Fig 2,3. Fig 2 indicates stress slow pattern by STM method where load is transferred *by means of truss action by forming diagonal strut and tension tie at bottom of girder. In above problem diagonal member carries 2415KN(compression) and horizontal member carries 1690KN. Same can be confirmed by FEM* model as indicated in Fig3. Where diagonal stress flow can be observed. Element at corners are lightly stressed and in STM method this material considered as infill material. Diagonal struts are bottle shaped for which busting steel needs to be provided and for tie at bottom, tension steel is to be provided.

b) A girder supporting floating column (carrying high moment)

Consider simply supported girder having 6M span with depth of 2M and for induction of moment into our plane stress model, vertical loads in the form of a couple has been applied. 1600 KN upward and downward load at 2m spacing has been applied as shown in fig 4. Net moment induction = (1600 X 1) + (1600 X 1) = 3200 T-M and net force of 0 KN

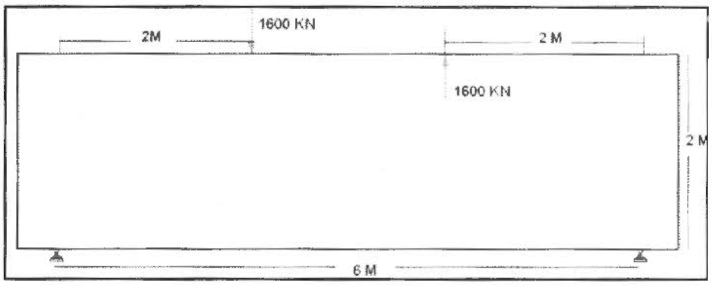


Fig.4

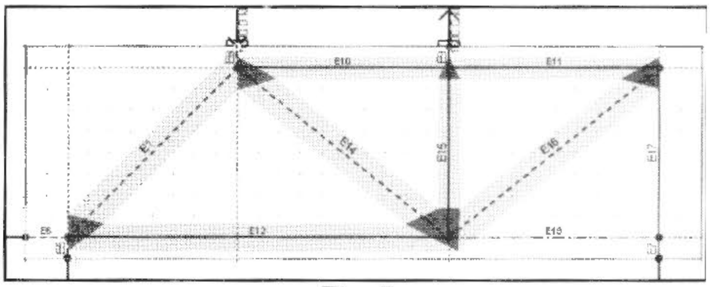


Fig.5

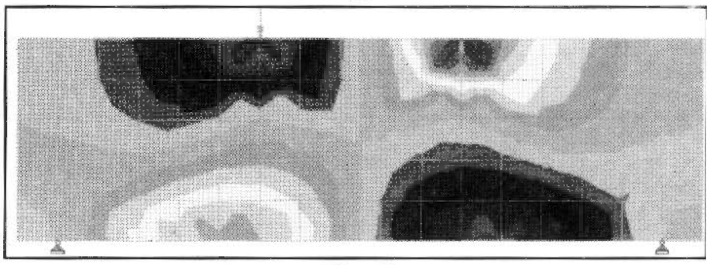


Fig.6

As it can be seen from Fig 5 of STM model and Fig 6 of FEM model, tensile forces get developed in right hand top and left hand bottom portion of the beam. For these tensile forces, we have to provide adequate reinforcement.

Example a and b can be combined for simulating girder supporting floating column having axial load and moment. In combined case of axial force + moment, net forces will be applied on girder i.e. compressive forces will be added and tensile force will deducted. E.g. if problem (a) and (b) are combined net force of 3200KN is applied at left and 0KN at right.

c) A single corbel

Consider a single corbel projecting from a 500MM X 700MM column supporting precast beam reaction 0.4M from the face of the column. The factored vertical load due to beam reaction is 1000KN.

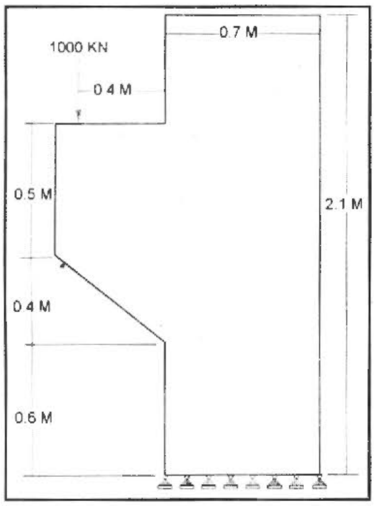


Fig.7

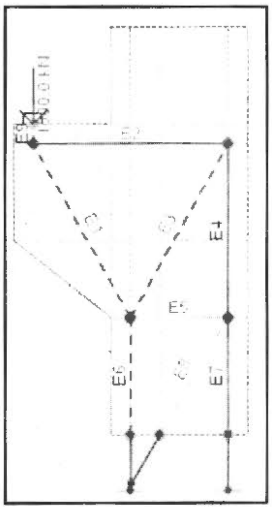


Fig.8

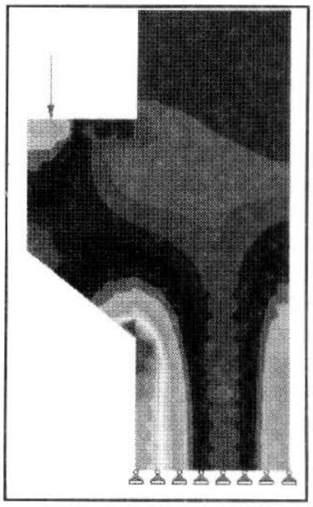


Fig.9

A bracket or corbel is a short member that cantilevers out from column or wall to support load. The corbel is generally built monolithically with column or wall. The term "corbel generally is restricted to cantilevers having shear span-to-depth ratio less than or equal to 1.

STM model (fig.8) stress flow can be clearly visualized. E2, E4, E7 carry 690KN, 1222KN, 1222KN tensile force respectively. where as E1, E3, E6 carries 1214KN, 1403KN, 2222KN compressive force respectively. E5 & E8 are truss stabilizing members; almost same stress flow pattern can be seen from FEM model (fig 9). Compressive stress flow pattern is observed on left hand side of column and tensile force on right hand portion.

d) A Dapped beam end

The ends of precast beams sometimes supported on an projection that is reduced in height, such detail is referred as a dapped end.

Consider dapped-beam end, which is to be designed to transmit a factored vertical load of 140KN. The beam is having span of 2.2M and depth of 0.75M with a width of 500 MM.

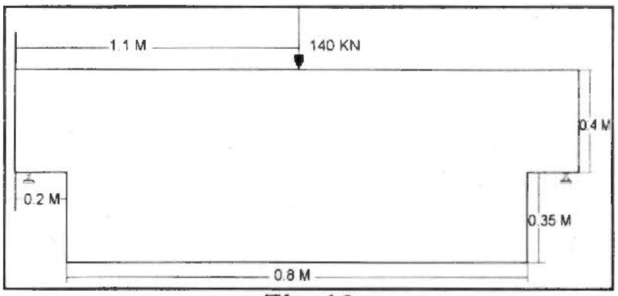


Fig.10

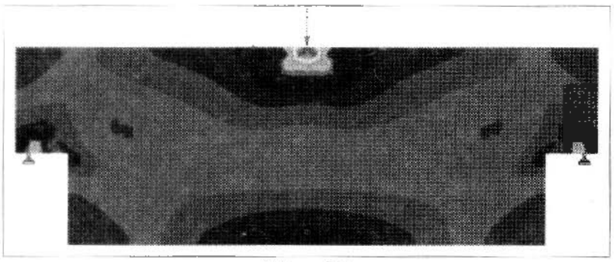


Fig.11

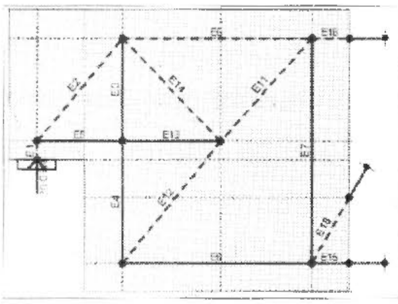


Fig.12

e) Pile cap supporting column carrying axial load and large moment

A pile cap transmits the column load to a series of piles, which in turn, transmit the load to a strong soil layer at some depth below the surface. Pile cap is a special case of a "deep beam" & can be idealized as a three-dimensional strut-and-tie model

Consider a column carrying axial load of 2000 KN and ant-clockwise moment of 9000 KN-M supported by four pile group.
Axial load of 2000 KN is sub-divided in four equal part of 500 KN each. For inducing anti-clockwise moment of 9000 KN-M downward force of 7500 KN is applied in left portion and 7500 KN upward force is applied in right portion. Net load is applied by superimposing above two load cases which is 8000 KN downward force in left portion and upward load of 7000 KN in right portion.

Axial load calculations

= 500 KN + 7500 KN

8000 KN (left portion downward)

= 500 KN – 7500 KN

-7000 KN (right portion upward)

Total Axial load calculations

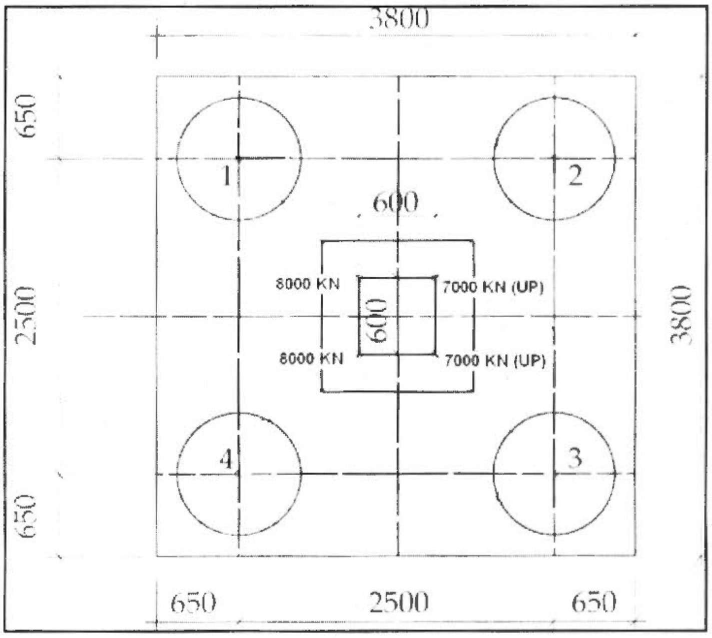


Fig.13 pile cap subjected to heavy lateral load

= [(8000 X 2) + (-7000 X 2)]

= 2000 KN

Anti-clockwise moment calculations

= [(8000 X 0.3) + (7000 X 0.3)] X 2
9000 KN-M (anti-clockwise)

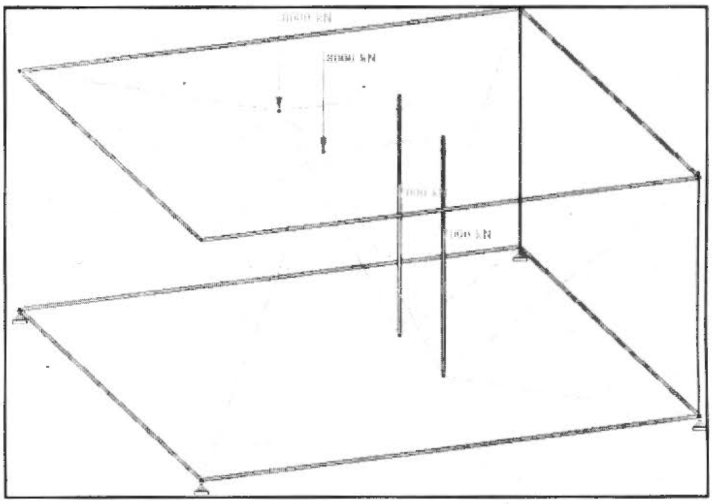


Fig14
Proposed 3-D strut-and-tie model

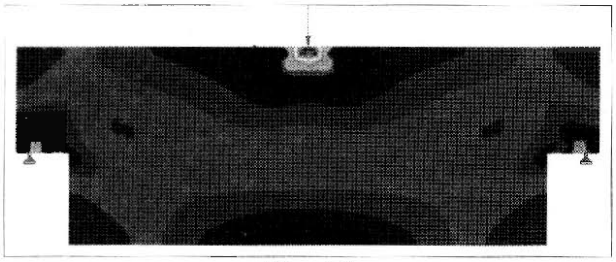


Fig.11

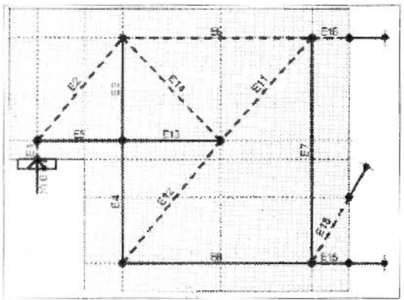


Fig.12

e) Pile cap supporting column carrying axial load and large moment

A pile cap transmits the column load to a series of piles, which in turn, transmit the load to a strong soil layer at some depth below the surface. Pile cap is a special case of a "deep beam" & can be idealized as a three-dimensional strut-and-tie model

Consider a column carrying axial load of 2000 KN and ant-clockwise moment of 9000 KN-M supported by four pile group.
Axial load of 2000 KN is sub-divided in four equal part of 500 KN each. For inducing anti-clockwise moment of 9000 KN-M downward force of 7500 KN is applied in left portion and 7500 KN upward force is applied in right portion. Net load is applied by superimposing above two load cases which is 8000 KN downward force in left portion and upward load of 7000 KN in right portion.

Axial load calculations

= 500 KN + 7500 KN

8000 KN (left portion downward)

= 500 KN – 7500 KN

-7000 KN (right portion upward)

Total Axial load calculations

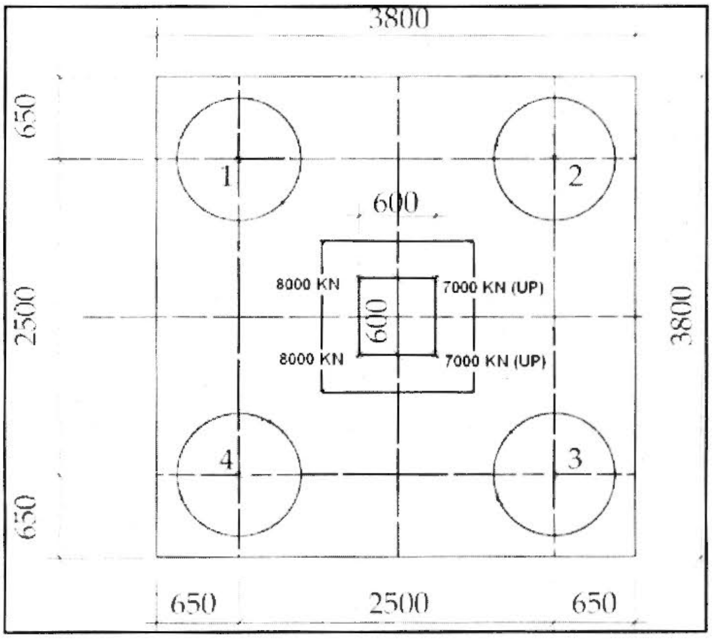


Fig.13 pile cap subjected to heavy lateral load

= [(8000 X 2) + (-7000 X 2)]

= 2000 KN

Anti-clockwise moment calculations

= [(8000 X 0.3) + (7000 X 0.3)] X 2
9000 KN-M (anti-clockwise)

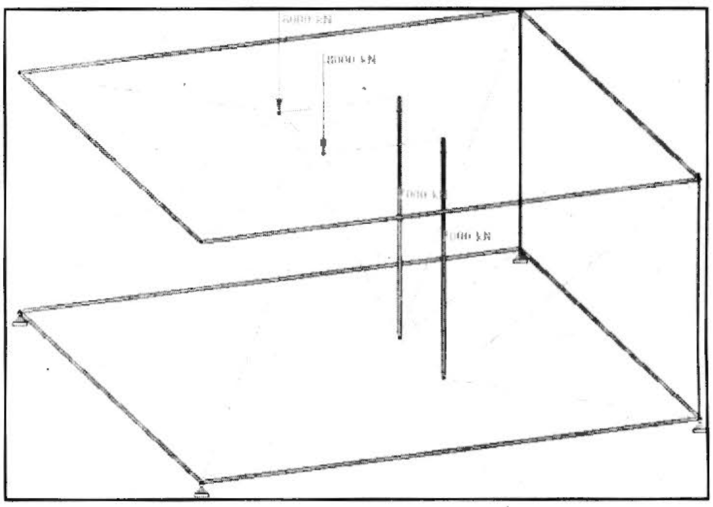


Fig14
Proposed 3-D strut-and-tie model

Publications available at ISSE head office

Title	Donation amount
Publications :	₹
• Design of Reinforced Concrete Structures for Earthquake Resistance	800
• Professional Services by Structural Design Consultant - Manual for Practice	200
Proceedings :	
• National Conference on Corrosion Controlled Structure in New Millenium	400
• Workshop on ISO-9001 for Construction Industry	200
• Brain Storming Session on Use of Speciality Products in Structures	250
• Workshop on Software Tools for Structural Design of Buildings with CD	550
• Workshop on Structural Audit	200
• Workshop on-Seismic Design of Building	200
• Workshop on Effective Use of Structural Software.	200
• Workshop on Effective Use of Structural Software - CD	150
• Workshop on Shear Walls in Highrise Buildings	200
• Seminar on Innovative Repair Materials / Chemicals	250
• Seminar on Foundations for Highrise Buildings	200
• Seminar on Structural Detailing in RCC Buildings	250
• Workshop on Pile Foundations	200
• Seminar on Pre-engineered Structures	200
• Workshop on Wind and Earthquake Load on Structures - CD	200

BE AN ISSE AUTHOR

Articles and technical papers are the heart of a technical journal. We invite you to write technical articles and papers for publication in the ISSE Journal. You may write about:

- An innovative concept or approach
- Proposed theoretical, computational or practical improvement on an existing concept
- An experimental study
- Guidelines and standards developed
- Compilation of rare/scattered information on the latest technological advances
- A case study: Challenges in design and construction
- Your viewpoint on current professional practices

While submitting your article for publication, please follow the guidelines given below:

- Page size: A4, Top, Bottom, Left and Right margins: 1", Font: Arial, 10 pt
- Max length of article: 5 pages including tables and figures
- The manuscript should contain the title of article and names, qualifications, designations, addresses and email addresses of the authors along with photograph.
- The matter should be relevant to the subject and should be organized in a logical flow. It may be divided into sections and sub-sections, if necessary.
- While, sketches and drawings should preferably be in Corel-draw, other appropriate formats are also acceptable. Photographs should be sharp and clear.
- Figures, photographs and tables should be numbered and should have captions.
- Notations, if used, should be clearly defined.
- Article should be sent by email to issemumbai@gmail.com

Articles may be reviewed and edited before publication.

Edited and published by N K Bhattacharyya for ISSE, C/o S G Dharmadhikari, 24 Pandit Niwas, 3rd floor S K Bole Marg, Dadar (W), Mumbai 400 028. Tel 91-22-24365240, Fax-91-22-24224096, e-mail issemumbai@gmail.com Web (www.isse.org.in) for private circulation and printed by S L Bengali, Bensan Printers, 15, Pandit Niwas, S K Bole Road, Dadar, Mumbai 400 028

REPORT ON ISSE ONE DAY WORKSHOP ON WIND AND EARTHQUAKE LOADS - 21MAY 2011

Indian Society of Structural Engineers (ISSE) in association with The Institution of Engineers India (IEI) arranged Lecture on ""WIND AND EARTHQUAKE LOADS ON STRUCTURES " On 21st May 2011 at the Auditorium of Institution of Engineers Maharashtra State Centre, Mahalaxmi, Mumbai.

Mr. S G Dharmadhikari, President ISSE welcomed all the delegates on behalf of ISSE and IEI.

Mr. Hemant Vadalkar coordinator of the workshop talked on importance of earthquake and wind loading in the structural analysis of high rise buildings.

Director Engineering Mr. A. G. Shrotri was the chief guest for the function. He promised MCGM's help to ISSE in implementing good engineering practices.

Mr. K. Suresh Kumar Managing Director RWID India made presentation on the wind load calculations based on IS875-Part3 and its limitations. He showed the comparison of International codes on wind loading. He explained intricacies of wind engineering applicable to the structural response of tall buildings, understanding of provision of IS: 875 Part3, 1987 Indian Standard for Wind Loads. He stressed on significance of wind tunnel testing of tall buildings, wind induced motions of cloud structure and key elements for structural design. He made it a point that cross wind oscillations, torsional effects are not accounted in IS875-Part 3. He expressed the need to have wind load applied in different directions with 100-60-60 combination. He discussed various international projects for which wind tunnel test was carried out...

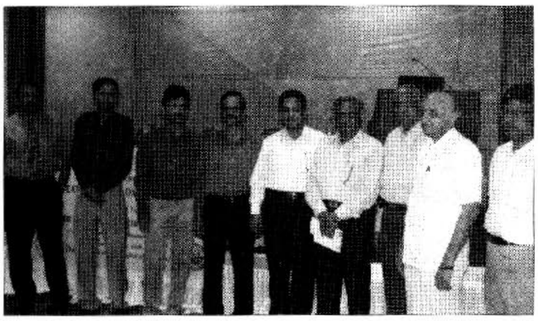
Dr. Mrs. M.A.Chakrabarti Head Structural Engineering Department VJTI talked on "Seismic load calculation as per IS 1893 – 2002 and response spectrum analysis in particular for tall buildings." She explained the philosophy of response spectrum analysis, method of combination of results of dynamic analysis, mode shapes and codal provision in a lucid manner.

Hemant Vadalkar, Consulting Engineer, Mumbai & Co-coordinator of the seminar made presentation on Wind load and Seismic loads practical application using STAADPro software. He explained the methods used in the software for seismic and wind load generations and discussed the sample output & interpretation. He also explained the common mistakes made by the users and how to avoid it. He stressed that results from any software must be checked by simple hand calculations and with engineering judgment.

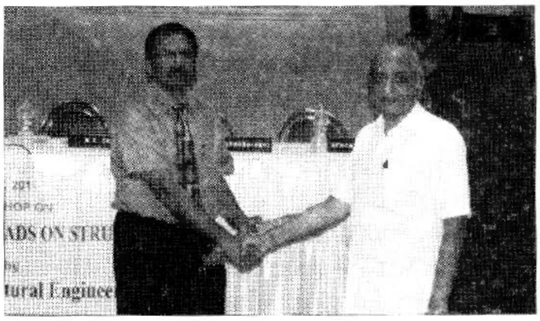
D. S. Joshi chaired question & answer session.

Mrs. Kirty Vadalkar conducted the program with her live comments. A CD containing reference literature and presentation lectures was distributed to all delegates.

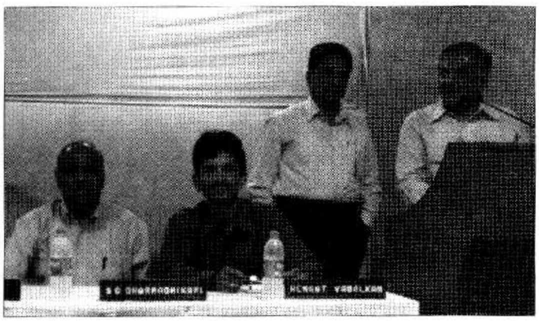
The response for the workshop was overwhelming. It was attended by about 150 delegates.



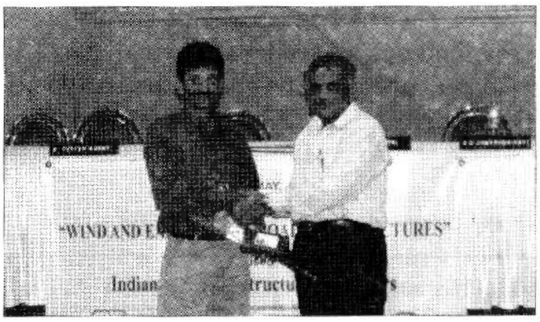
ISSE Team & Dignitaries



D. S. Joshi Welcoming Dr. Suresh Kumar



S. H. Jain Introducing A. G. Shrotri



S. H. Jain Felicitating Hemant Vadalkar

Banner

Inauguration

Dr. K. Sureshkumar

Dr. M. A. Chakrabarti

Hemant Vadalkar

Audience

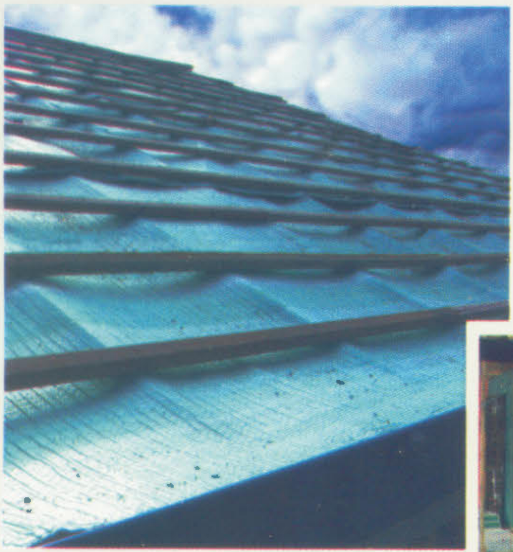
A. G. Shrotri

Kirty Vadalkar

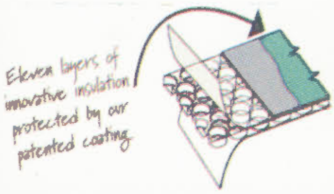
Green Insulation
THE INTELLIGENT SOLUTION

Reflecta-Range™

Leading the way in protection against oxidisation and corrosion



The product Structure



The REFLECTA-RANGE™ of Products

- REFLECTA-SHIELD™
- REFLECTA-CELL®
- REFLECTA-GUARD™

Green Insulation™ products are suitable for
Residential • Commercial • Industrial Building applications

✓ Energy Saving ✓ Lasting Comfort ✓ Safe and Green

Authorised Distributor :
Roofex
Suite No.19, 1st Floor,
Plot No. 3A, Udyog Nagar,
Goregaon (W), Mumbai - 400 062.
Tel.: 91-22-2876 6563 / 2876 8881
Fax : 91-22-2876 3244
E-mail : hitesh@roofex.in
Web : www.roofex.co.in