STRUCTURAL ENGINEERING

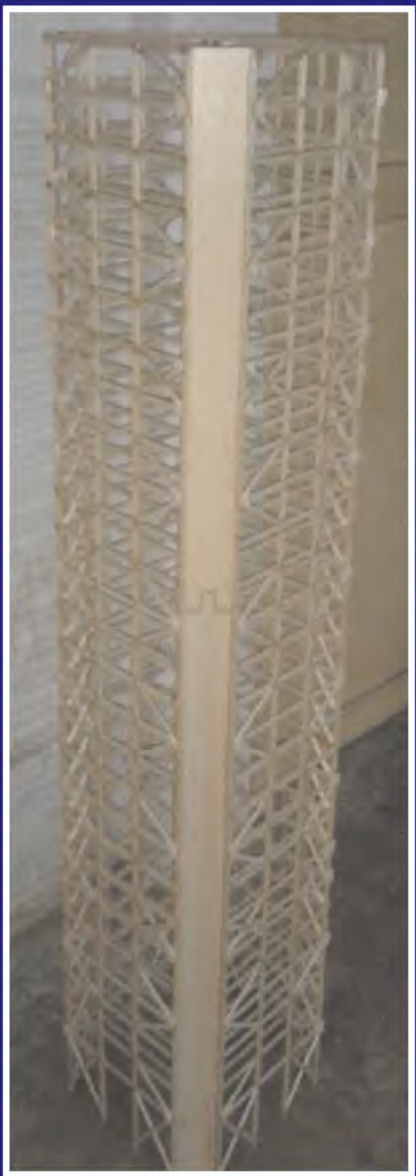
QUARTERLY JOURNAL OF

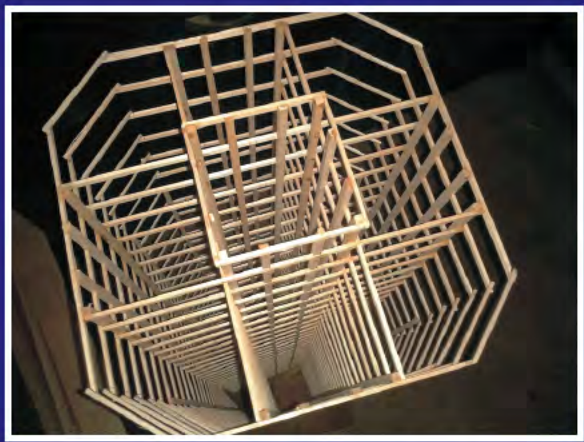
INDIAN SOCIETY
OF
STRUCTURAL ENGINEERS

ISSE

VOLUME 14-1

Jan-Feb-Mar-2012





Earthquake Resistant Design of Strucure &
Experimenal Analysis of Scaled Model
(See Page 19 inside)

LET US BUILD A STRONG STRUCTURE OF INDIAN SOCIETY

STRUCTURAL ENGINEERS

QUARTERLY JOURNAL

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

ISSE

VOLUME 14-1, JAN-FEB-MAR 2012

Head Office : C/O S. G. Dharmadhikari, 24, Pandit Niwas, 3rd Floor, S. K. Bole Road, Dadar, (W), Mumbai 400 028 • Tel. 91-22 24365240 • 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 under 80-G

Contents

Editor : **Hemant Vadalkar**
N K Bhattacharyya

Fraternity News

WELCOME TO NEW MEMBERS

(Oct-Nov-Dec 2011)

M- 1167 Ms. Shital Ajit Surwade

M- 1168 Mr. Rajeev Gupta

M- 1169 Mr.Avinash Kail

M- 1170 Mr.Majid Dhokle

M- 1171 Mr.Mohd Iqbal Ghufran

M- 1172 Mr.Jagadish Patil

M- 1173 Mr.Sushil Chaudhary

M- 1174 Mr.Amol M Sawant

M- 1175 Mr.Amarnathr Boraiah

M- 1176 Mr.Shaik Khadar Basha

M- 1177 Mr. Ajit Gijare

M- 1178 Mr.Milind Bhoot

Patrons : 29	Organisation Member : 20	Sponsors : 8
Members : 1178	Junior Members : 11	

TOTAL STRENGTH 1246

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 organizations 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.

REPORT ON ISSE'S 14TH FOUNDATION DAY CELEBRATION

Hemant Vadalkar

Indian Society of Structural Engineers (ISSE) celebrated its 14th foundation day on 3rd Feb 2012 at Rachana Sansad College Hall, Prabhadevi, Mumbai.

President Shri S.G.Dharmadhikari welcomed all the guests and members. He touched upon the history of ISSE and its active involvement in arranging the lectures , seminars and workshops for civil and structural engineers. Secretary K.L.Savla presented the activity report of ISSE during the last one year elaborating the lectures and workshops conducted by ISSE. Some of the important events organised by ISSE are as follows -

1) One day seminar on "Pre-engineered Constructions" On 29th Jan 2011

2) Lecture by Seshadri Srinivasan CONSULTING ENGINEER, UK on"The Importance of Concept Design in the Design and Construction of Bridges" on 25th March 2011.

3) Lecture by Mr.Robert Azimi,Head for Global Operations, Sans N. America of M/s. FM Approval on "FM Global's Approach to Safe Design Practice to Limit Property Losses" on 11 May 2011.

4) One day workshop on "WIND AND EARTHQUAKE LOADS ON STRUCTURES" on 21st MAY, 2011

Secretary Mr. Savla congratulated ISSE advisory trustee Shri Hemant Vadalkar for writing series of articles on Structural Engineering in local marathi news paper Loksatta. Secretary appealed to all the members for their active participation in ISSE activities.

The president and Secretary honoured the authors of ISSE quarterly journal for their valuable contribution in the form of technical articles. ISSE felicitated the authors by presenting a memento and a certificate.

Mr. Hemant Vadalkar appealed to all practicing consulting engineers to share their knowledge and experience through ISSE platform like lectures, seminars and journal. He also announced that the new ISSE website is operational and members can visit the site to know about all activities of ISSE and technical information.

A technical lecture on "Post-Tensioning Technique and its implementation: Good Design and Construction Practices in PT Structures" was delivered by overseas experts Mr.Khalid Rabadi and Osama Masarweh. They touched upon very important aspect of post tension slab construction in the modern buildings and provisions in the different codes of practices. The function was sponsored by Freyssinet Menard India Pvt Ltd.

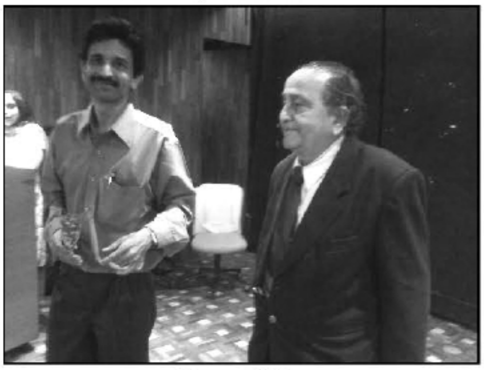
ISSE team welcomed the new President Prof. G.B.Choudhari and new Secretary P. B. Dandekar.

The program was coordinated by Mrs. Kirty Vadalkar. It was attended by about 80 engineers.

ISSE Team

M.C. Bhide
Felicitating New
Secretary P. B. Dandekar

M. C. Bhide
Felicitating New Precedent
Prof. G. B. Chaudhary



M. C. Bhide
Felicitating H.S. Vadalkar

M.C. Bhide Felicitating
Treasurer M.M. Nandgaonkar

Prof. Sundaram & M. C. Bhide

K. L. Sawla Felicitating Mayuri Patil

S. G. Dharmadikari Felicitating
A.B. Karnik

S. G. Dharmadikari Felicitating
Vivek Abhyankar

WIND ENGINEERING FOR DESIGN OF STRUCTURES - FUNDAMENTALS

K. Suresh Kumar

(1) **What is wind engineering?**

Wind engineering is a niche field under the umbrella of Civil Engineering. Many times, wind engineering is being misunderstood as wind energy in India. On the other hand, wind engineering is a unique part of engineering where the impact of wind on structures and its environment is being studied. More specifically related to buildings, wind loads on claddings are required for the selection of the cladding systems and wind loads on the structural frames are required for the design of beams, columns, lateral bracings and foundations.

(2) **Do we have to consider wind loads for structural design?**

Yes. Structural design of lateral loading systems of structures should consider lateral loading forces such as wind and earthquake. The final design will be based on one of these lateral forces, whichever is governing, along with dead load and live load. Considering the rare chance of simultaneous occurrence of both earthquake and high wind, both earthquake and wind loads won't be combined together in any structural design.

(3) **What are the fundamental differences between wind and earthquake forces?**

Earthquake acts only for a very short duration of the order of 0.5 min, while wind acts for long duration of the order of 15min to many hours. Earthquake acts at the base and the force is fully correlated over the height of the structure. On the other hand, wind acts on the exposed surface of the structure and generally, wind loads are correlated over patches of the surface area. Earthquake energy is concentrated at higher frequencies and this will induce serious damage to a similar frequency structure such as low buildings. Wind energy is concentrated at lower frequencies and this will induce serious damage to a similar frequency structure such as tall buildings. As per my consulting experience, wind in general governs the design when buildings are above 150 m height. When buildings grow taller, they become flexible and as a result, moving away from the high frequency earthquake waves.

(4) **What are the indicators of potential severe wind effects on a structure?**

(a) Higher height/long span, (b) High slenderness in vertical or horizontal direction (e.g. tall buildings, long span roofs of airports/stadiums, long span bridges), (c) light weight, (d) blunt shape, and (e) flexibility.

(5) **When should the design team be thinking about wind design?**

I would say on the very first day of design if the structure in mind is iconic, slender and light weight. One can make wonders by increasing the wind resistance of the structures by choosing the right geometry, orientation and structural systems. In case of typical structures, design team shall consider wind design when the external architectural dimensions are crystallized.

(6) **Where do we start with the wind design?**

Preliminary wind load calculations can be carried out using the Indian standard, IS:875 (Part 3: Wind Loads). Please note that the code is for preliminary design and it doesn't cover all the cases. Limitations of the code are also mentioned in the code itself.

(7) **What are the limitations of IS:875 (Part 3:wind Loads)?**

As per IS:875 Part 3 1987 (Page 5), "Note 1 – This standard does not apply to buildings and structures with unconventional shapes, unusual locations, and abnormal environmental conditions that have not been covered in this code. Special investigations are necessary in such cases to establish wind loads and their effects. Wind tunnel studies may also be required in such situations. Note 2 – In the case of tall structures with unsymmetrical geometry, the designs may have to be checked for torsional effects due to wind pressure. (Page 48) Note 9- In assessing wind loads due to such dynamic phenomenon as galloping, flutter and ovalling, the required information is not available either in the references of Note 8 or other literature, specialist advise shall be sought, including experiments on models in wind tunnels.

(8) **What is wind tunnel testing?**

In wind tunnel studies, scaled models of structures are subjected to scaled atmospheric wind in a controlled laboratory set-up. Then sensors installed on the model can measure the physical quantities of interest such as shear, moment, pressure etc. Later in the analysis, these model scale quantities are converted to prototype using model scale laws.

(9) **Are the wind tunnel tests done at expected wind speeds at site?**

No. Similar to the scaling of the building dimensions to prepare a model to test in the wind tunnel, the actual wind speed at the site is also scaled down in the wind tunnel. Typically, the speeds are scaled down by a factor of 3 to 5. Interestingly enough, the scaled models probably cannot withstand the high speed at the site even if we wanted to simulate such a speed in the tunnel.

(10) **What are typical scales of the wind tunnel models?**

The wind tunnel models are typically 300 to 500 times smaller than the prototype.

(11) **How long should we carry out the measurements?**

Once again, similar to length and speed, time is also scaled in the tunnel. The time scale can be derived once the length and velocity scales are known. Typically, the wind tunnel measurements are roughly 80 to 160 times shorter than the full-scale measurements. When wind is blowing in real life situation, the response of the structure should be measured for duration anywhere between 15 min to few hours in order to capture the structural interaction with wind. In model scale, this amounts to anywhere between few seconds to few minutes.

(12) **What is the information required for tunnel test?**

(a) Architectural drawings in soft copy (all floor plans, elevations, sections and roof details), (b) Site plan and surrounding building details for a radius of 500m from the study site, and (c) structural information.

(13) **What more do wind tunnel tests cover than codal provisions?**

Wind tunnel tests account for the actual building geometry, local climate and surrounding details and this leads to cost-effective and accurate wind loading on structures. In general, Code analytical methods are helpful for preliminary design and for simple situations, but provide conservative wind loads in most cases; underestimating in others. Presently, wind tunnel model studies offer the best estimate of the wind loading acting on structures.

(14) **When to do wind tunnel test?**

(a) When the structure has an unusual or irregular geometric plan, form and/or shape, (b) when the structure is the midst of significant neighboring buildings, (c) when the buildings are above 40 storeys (~150m above grade) and (d) when the slenderness ratio (height/width) exceeds 5.

(15) **Do we have to carry out gust factor analysis in the code after the wind tunnel tests? Does the wind tunnel test cover the effects of gust?**

No. The wind tunnel tests do cover the gust effects. The gust effects are contributed by the upcoming fetch roughness, the immediate surrounding buildings, and the body induced turbulence generated by the study building itself. In codal approach, the effects of upwind fetch roughness and the body induced turbulence are included through a gust factor.

(16) **Does the wind tunnel test follow the same way as code with regard to gust factor?**

No. The wind tunnel accounts for the gust effects by physically simulating the upwind fetch roughness, immediate surroundings and body induced turbulence, and subsequently these gust effects will be captured in the measured quantities such as forces, moments and motions. Therefore, wind tunnel test normally estimate the peak load first and then we can back-calculate the gust factor out of the peak and mean loads. In contrast, in the codes, generally gust factor is calculated first followed by the peak load estimation.

(17) **Do the tunnel results include local wind directionality? Do the code loads include directionality?**

Yes. The tunnel results include wind directionality. The code loads in general do not include directionality. In few codes, wind directionality is defined through a directionality factor.

(18) **What is wind directionality? And its use in design?**

Wind directionality represents the probability of the design wind speed blowing for a sweep of angles between 0 to 360 degrees. In other words, for a given location, the design wind speed, based on a constant probability of exceedance (i.e. risk factor) blowing from various different directions, can be different. Therefore, after the wind tunnel test, during the analysis stage, the non-dimensional response of the tower will be combined with the design wind speed based on a constant risk factor for each angle of attack. This way one could take advantage of the actual design wind speed from each angle of attack at the site instead of using the constant design wind speed from the code for all wind angles. This will potentially reduce the loads on a structure except when the non-dimensional response and the wind speed peak at the same wind angle.

(19) **Do the wind tunnel results are at a particular risk level?**

Yes. Note that the wind speed/wind loads are fluctuating and they are not a deterministic quantity. Therefore, the magnitudes of wind speeds/wind loads can only be determined with an associated probability of exceedance or risk. In codes as well as in the wind tunnels, this risk is associated with the wind speed. In general, the wind loads are predicted for 50-yr return period (i.e. probability of exceedance in any given year is 1/50=0.02) wind speed. For serviceability checks, the loads/motions corresponding to a much lower return period of the order of 10 to 20 is adequate considering the fact that this is not a safety check more of a comfort check.

(20) **Do we have to apply factor of safety on the wind tunnel loads? Philosophy?**

Yes. The factor of safety from corresponding codes should be applied. This factor will cover all the uncertainties related to wind speed predictions and wind tunnel measurements. Alternatively, factor of safety will elevate the 50-year predicted loads to an ultimate limit state load.

(21) **Is the wind tunnel test economically viable & is this really required?**

In general, cost of a wind tunnel study is dwarfed by the reduction in cladding and structural costs, which often add up to crores of rupees. In addition to the potential cost savings and accurate results, wind tunnel studies confirm that the architect's vision can be safely built and elevate litigation protection. Last, but not the least, most of the Codes themselves recommends wind tunnel testing for complex structures.

(22) **Am I doing wind tunnel tests for saving on construction?**

No. Wind tunnel tests are mainly carried out to accurately evaluate the wind loads/motions of the structure. This will help in designing the structure for safety and serviceability. Sometimes saving is a byproduct when the wind tunnel derived loads are lower than those based on codes. This will result in enormous savings on construction costs of foundations and superstructure.

(23) **Is it useful to carry out wind tunnel test once the foundation is over?**

Not advisable at this stage. Probably the foundation is already overdesigned by now. The only use at this stage is probably to check the superstructure design.

(24) **What can we do if there is a design change after the tunnel test?**

If this is an internal structural change, then we need not repeat the wind tunnel test. Instead, we can carry out a reanalysis of the existing wind tunnel data with the new structural properties and come up with updated loads and accelerations. On the other hand, if the changes are on the external architecture, the wind tunnel consultancy team would have to review the drawings and take a call on the repeat test. If the architectural changes are minor (such as few floors increase or decrease), then a review and desktop assessment of the changes are sufficient for design instead of repeating the test.

(25) **How do we account for the surrounding building scenario that may change over the years?**

Generally, the surrounding will build up over the years and this will reduce loading on any given building by blocking the wind rather than increase. However, in developing nations where cities are just beginning to grow vertically, unusual structures coming closer to the existing structure may induce higher loading on the existing structure when they are at downstream. Note that this higher loading will be induced at only few wind angles and this increase will generally be covered by the design safety factor. During wind tunnel testing, this issue will be partially addressed in the form of phase testing to accommodate different times of construction of the same development and/or adjacent upcoming development.

(26) **How many different types of common wind tunnel tests?**

(a) Local pressure test: local pressure measurements on scaled static models instrumented with pressure taps, (b) High Frequency Force Balance (HFFB) Test: measurement of overall wind loads on scaled high frequency static models using balance, and (c) High Frequency Pressure Integration (HFPI): Simultaneous pressure measurements on scaled static models instrumented with many pressure taps and subsequent spatial or time averaging of simultaneously acting local pressures to obtain overall wind loads.

(27) **What is an aeroelastic study and when this is required?**

In aeroelastic studies, the structural mass, stiffness and damping will be appropriately modeled in comparison to the rigid model tests mentioned above. Inertial effects due to structural motions will be measured in aeroelastic model study while they are computed in rigid model tests. For structures which are considerably slender, tall or long span, aeroelastic effects can be significant and under this circumstance aeroelastic study is recommended. However, typical structures do not require such an elaborate model study instead simple rigid model study would be sufficient.

(28) **How do wind tunnel results compare with code results?**

Majority of the time, wind tunnel results are lower than code (means international codes here) based values since the structures are with unconventional shapes, surrounded by other structures and subjected to design wind speeds only from few wind angles. However, in some cases, the wind tunnel results can be also much higher than the code based values for various reasons.

(29) **What are the reasons for higher values from wind tunnel?**

This can be due to (a) peculiar shape, (b) interference from an adjacent building and (c) complex structural systems.

(30) **What can we do when the wind tunnel results are lower than code?**

Please use the wind tunnel results for the design as this is the most accurate results accounting for the actual geometry, surroundings and the structural system. It is a wrong perception that the code calculations provide the ultimate answer. Most code users are unaware that the standard is derived from a set of wind tunnel experiments of buildings with simple geometries.

(31) **Any limit on lower loads from wind tunnel?**

Wind tunnel results can be sometimes very low compared to the code due to significant surroundings and/or geometry. In such cases engineering judgment should be applied while recommending loads for design. Typically, lower cutoff of loads may get applied based on code loads. If the reduction of loads is confirmed due to geometry, ASCE code is allowing up to 50% reduction for structural loads and up to 65% for cladding loads. However, the load reduction is due to immediate surroundings, code is allowing only up to 80% limit.

(32) **What can we do when the wind tunnel results are higher than code?**

Please use the wind tunnel results for the design as this is the most accurate results accounting for the actual geometry, surroundings and the structural system.

(33) **What can we do when the wind tunnel results are lower?**

This means the structure is overdesigned and there is a possibility of optimizing section sizes. The structural engineer can run through this exercise of tweaking the section sizes and thereby optimizing the structure. A few iterations of reanalysis with the same wind tunnel data are required as a part of this exercise.

(34) **How does IS:875 Part 3 compare with other international codes?**

Our code is 23 years old and this needs to be updated sooner than later considering the construction revolution in India. Our code must be used with caution when we use it for cladding design since the corner pressures are underestimated. Without the presence of cross wind domination, structural load estimation seems at par with other international codes. Many other subject matter from the IS:875 standard such as wind speed map, loading on roof and other structures requires revision as well.

(35) **Wind speed map: how reliable they are? Any other suggestions on wind measurements at the airport?**

Based on our experience so far in India, robust measurement systems as well as meaningful measurements are essential for better understanding of the wind climate of India. We have noticed that local wind speed for many cities are in general conservative in the wind speed map of the IS:875 Part 3 code compared to our own predictions based on the airport data. The speeds are conservatively put in the map probably to cover up the uncertainty in the wind speed measurements. More sophisticated equipments should be used for wind speed and direction measurements. It is noticed that the wind directionality is typically measured every three hour and this is of no use for the wind engineering. Continuous measurement of the wind speed and direction data at a frequency of about 10 Hz is required for obtaining better statistics of the wind speed and direction data for any city.

(36) **Do structures oscillate due to the impact of wind?**

Yes. Mostly human eye can't see this especially in case of tall buildings. Structural engineers are interested in deflection calculations since the design has to satisfy the deflection criterion typically in the range of Height/500. But as far as human perception is concerned, wind engineers are interested in determining the acceleration rather than deflection. This is due to the fact that a particular deflection is felt considerably in a stiffer building than in a flexible building. In other words, a particular oscillation magnitude at a longer period is felt less than at a shorter period.

(37) **Are the oscillations are large enough to be perceived by the building occupants?**

Though naked eye can't see the motions but human beings can feel when they are crossing certain limits. In contrary to the expected bouncing in an airplane caused by turbulence, human beings are not expecting any noticeable motions for instance on the top of a high rise building. So basically human perception and expectation are at high levels when they are using any structure such as tall buildings, bridges etc. The acceleration is generally described in literature either using root-mean-square (RMS) value or the peak. Research indicates that people first begin to perceive accelerations when they reach about 5 milli-g (where milli-g is 1/1000 of the acceleration of gravity). For a building of time period 3 seconds, the deflection corresponding to the perceivable acceleration of 5milli-g is about 1 cm. The corresponding deflection for a flexible building of 6 seconds time period is found to be about 4.5 cm. This shows that more deflection is allowed for a flexible building since they are oscillating at a longer time period. In summary not just the deflection but how quickly this is occurring is equally important when we talk about human perception of motions.

(38) Is it absolutely necessary to account for structural motions during the design of the structure?

Yes. For serviceability design of the structure, both deflection as well as acceleration has to be within limit.

(39) Is codal estimation of structural motions sufficient for the design?

First of all very few codes address this issue. Structural motions especially accelerations are quite difficult to generalize to put in the code for a wide variety of structures.

(40) Can the structural motions be accounted in wind tunnel study?

Yes. Wind tunnel study can determine the accelerations and typically this is done at the topmost occupied floor for a tall building and compared against the available criterion. Similar to translational motion, torsional motion is also equally important to be assessed in connection with occupant comfort. Torsional motions will create visuals cues among the occupants and they are assessed in the literature using peak torsional velocity in milli-rad/sec.

(41) Can a range be defined where the structural motions are acceptable or tolerable by the occupants?

Based on numerous research in the past, few codes/organizations such as National Building Code of Canada, International Organization for Standardization (ISO) and Council on Tall Buildings and Urban Habitat (CTBUH) recommend acceleration criterion for accelerations. Note also that the acceleration criterion is different for different structures such as tall buildings, bridges. Based on discussions between RWDI and the designers of numerous high-rise towers, RWDI uses the criterion of 15 to 18 milli-g for a residential building. For torsional velocity limits, RWDI uses the available CTBUH criteria of 1.5 milli-rad/sec and 3.0 milli-rad/sec corresponding to 1- and 10-year return periods respectively.

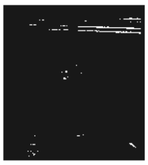
(42) Is the acceptable range of motions same for all types of buildings?

No. The acceleration limits can be relaxed considering the type of occupancy in a building. For instance in case of office buildings, the acceleration limits can be relaxed based on (a) its limited access during nights and weekends, and (b) the psychological notion that this is a place you just work along with other employees. RWDI uses the criterion of 20 to 25 milli-g as the acceleration limit for an office building, which is similar to CTBUH recommendation. Please note that consequence of higher accelerations is an increased likelihood of occupant discomfort, rather than an issue of life safety.

(43) What do we do when the structural motions are above the acceptable range?

If the structural motions are marginally above criterion, then very likely through structural tweaks one could manage to bring the motion within the criterion. This means the structural stiffness and/or mass increase is required. If the motions are considerably above criterion, then probably provision of supplementary damping along with structural modifications are required to bring down the motions. When the motions are far in excess of the criterion, then one may have to relook at the geometrical shape as well.

About the Author :



K. Suresh Kumar, Ph.D., P.Eng.,
M.ASCE Principal/ RWDI (India)
Managing Director RWDI India,
Email : suresh.kumar@rwdi.com
www.rwdi.com

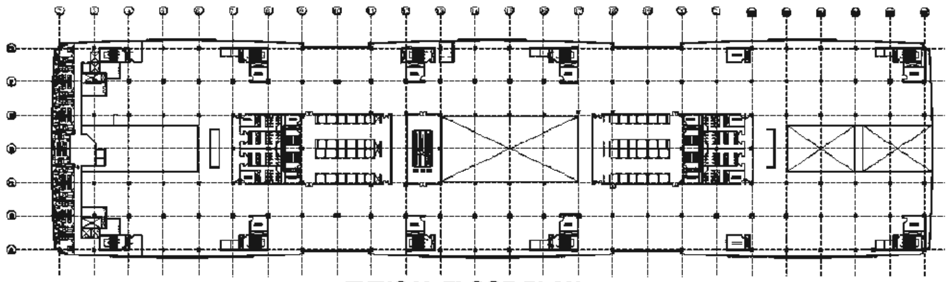
STRUCTURAL DESIGN OF THE ICICI BANK REGIONAL HUB, HYDERABAD

Girish Dravid

INTRODUCTION:

This large office complex has been recently completed in the IT hub of Gachibowli in Hyderabad for ICICI Bank. The plot is 300 m long and 80 m wide. A typical floor plate is 260 m x 64 m making it one of the largest office spaces in India today. The entire built up area is four lakh square meters and accommodates 22,000 employees in one shift at a luxuriously large ratio of area to a person compared to normal international standards! This is said to be the largest single office building owned and used by a single owner for activities for a single business field in India and the 18th largest building in the world in terms of floor area available in one single building!

ARCHITECTURAL PLANNING



TYPICAL FLOOR PLAN

Designed by Architect Hafeez Contractor, there are three full basements and partial fourth one, above which there are four podium levels and then twelve typical office floors in this building.

Architectural highlights of this building include a six floor high atrium in the entrance lobby. The building is separated by two expansion joints, forming three blocks served by individual lift banks. The three blocks are aesthetically connected by elevation features such as hanging gardens, tree windows and bridges. A large sloping garden stretching from end to end of the plot, rising from the second basement level at one end to the ground level at the other, supported on the sub-structure enhances the experience while entering the premises.

The usage areas of the building include car parking in basements, huge entrance lobbies, public spaces that house cafes, restaurants, gymnasia, meeting places, lounges etc. , banking halls, office spaces, training centre, auditoria, guest house cum dormitory for trainees arriving from outstation, vaults, call centre, data centre and other banking related activities.

STRUCTURAL SCHEME:

In the times of economic surge and rapid growth in Financial and IT sector in India, ICICI Bank set a goal for the building to be completed in just 18 months, not to miss the advantage of the situation. In order to achieve this seemingly impossible objective, it was decided to construct the building as a composite structure. A very trimmed schedule was chalked out in which as soon as the architectural planning was frozen to suit the client's needs, the general grids were finalized for the excavation and the construction of the basement to commence. The super structure above ground was planned in structural steel columns, beams and deck sheets, while the sub-structure and four podium slabs were planned to be in Post-Tensioned and Reinforced Concrete. The idea was to achieve substantial progress on the project during the period for ordering and procuring huge quantities of structural steel members that were required for the construction of the super-structure above podium level. Due to sequential construction requirement for RCC/ PT slabs, time duration for basement and podia was fixed as nearly 9 months, leaving only 9 months construction period available for the 12 typical structural steel floors in the tower.

The Gachibowli terrain is quite interesting. The surface is strewn with huge boulders, almost like a moraine. Below lies very stiff layer of highly weathered rock and at about six meters downwards it is a mass of very hard rock. At the site of this building, the rock level dipped along the length of the plot. The rock strata offered excellent bearing capacity for the foundation design to carry huge loads, to the tune of almost 4000 M.Tons on some of the columns. However, it was difficult to break the rock to accommodate the basements. Hence, the number of basements were adjusted to suit the dip in the rock level to avail of the high SBC but to avoid excessive excavation in hard rock.

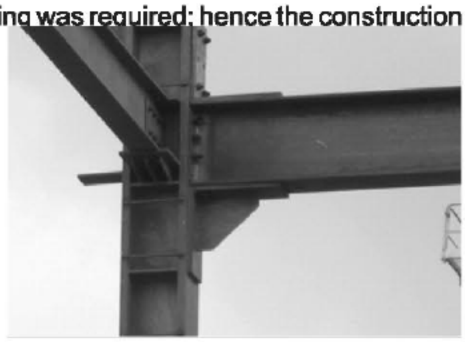
The impressive atrium in the entrance lobby is almost six floors high. This high volume space was created by using 30 m spanning transfer trusses which in turn carried the load of the upper 12 floors. Due to lifting capacity constraints, these trusses were fabricated in three parts each and then assembled in-situ with the help of holding cranes. There are other transfer mechanisms in the building, especially in the auditoria and in the hotel section, where large column free spans were needed and the column grids changed due to hotel room configurations. These transfers of columns were effected with fabricated plate girders spanning 10 to 30 m.

TRANSFER GIRDERS DURING ERECTION

DURING CONSTRUCTION

Typical Floor grid was fixed as 10.9 m x 10.6 m with secondary beams spaced at 3.55 m c/c. In order to save time for fabrication, readymade rolled British Standard sections were imported. Deck slab of overall depth of 170 mm was adopted using Corus Deckspan Comflor 80. The secondary beams were designed with composite action. Primary beam spans were 10.6 m and designed with partial moment connections. The overall structural depth turned out to be 785 mm, within the floor to floor height of 4.0 m, which was accepted by architects and MEP consultants.

In order to achieve an approximate construction cycle at 6-7 days, an aggressive construction sequence for superstructure was adopted. While columns of height 8 to 12 m were erected, beams were erected at 2 or 3 levels almost simultaneously. Pre-cut deck-sheets were laid over the beams, shear connectors welded and reinforcement tied. No propping was required; hence the construction cycle was



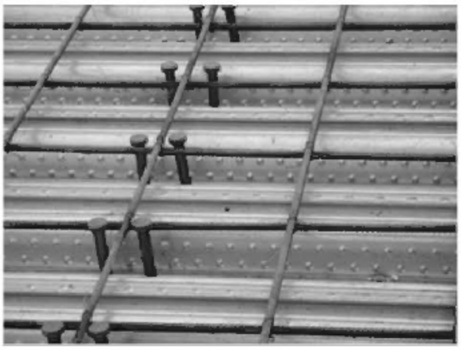
BEAM COLUMN CONNECTIONS

BEAM COLUMN CONNECTIONS

practically complete with the laying of deck-sheets. Concreting was the last activity, but was out of the critical path.

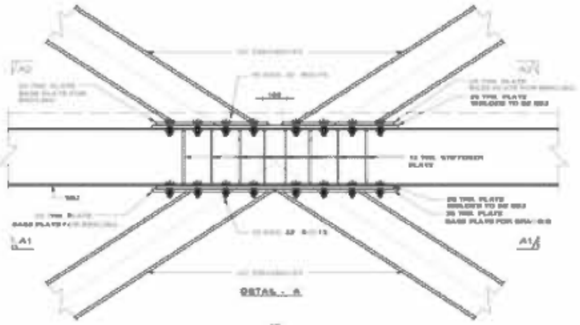
ComFlor 80 deck sheets were chosen after comparing it with the design using Comflor 60 with more secondary beams. Composite action ensured saving in secondary beam weight. Use of deck sheets eliminated the need of shuttering system, relieving the contractor of space and storage requirements for the formwork and staging materials. Since there was no propping, services and other installations could start immediately after curing of concrete above deck sheets. The deck-sheet layout drawing was prepared based on structural drawing. After our approval the deck-sheets were ordered as cut to size. The deck-sheets were received on site with appropriate marking. Geometrically no further check was required on site as the entire cutting was pre-planned on the drawings. Non-standard areas to an extent of about 5 to 10% were dealt with on the site itself. Since length of sheet was one full grid, local inaccuracies in steel erection could be covered at the edges on beams.

TRANSFER GIRDERS AFTER ERECTION



DECK SHEETS WITH STUDS

BRACING CONNECTIONS



BRACING CONNECTION DETAIL

Columns were encased in structural concrete to reduce steel consumption and also to protect them from fire and corrosion. Beams and bracings were coated with vermiculite. Exposed structural steel members, wherever used as architectural expressions, were applied with intumescent fire retarding paints. The concrete lift cores, which also acted as shear wall elements, were slip formed in advance of the structural steel erection.

In all, about 30,000 M. Tons of structural steel was used and about 200,000 sq.m of deck sheets were used in this building.

CO-ORDINATION :

Although Sterling has been involved in many structural steel building designs, for many other agencies that participated in the planning and design of the ICICI building, this was a new experience. Our team assumed a leader's role in explaining the possibilities and constraints of accommodating services and interiors, sequence of construction, temporary props and bracings and other quality related matters to designers, engineers and project managers. Fortunately, the discipline to be followed in the steel design and construction was understood and grasped by the architects, MEP engineers, façade contractors and the general contractor. Although architects threw many challenges by introducing pleasing architectural features generally associated with the usual concrete buildings, all of them could be accommodated without compromising the principles of steel construction.

QUALITY CONTROL :

Sterling played an extremely important part in ensuring quality in all aspects of design, drawings and even construction. Along with use of sophisticated analysis softwares for the whole structure, composite members, decksheets, slabs etc., extensive optimization exercises were carried out to arrive at the best economical grid and secondary beam spacing. All the time, consideration was given to the requirement of fast construction. Hence, the emphasis was on the minimum number of units to be erected – columns, beams, deck sheets etc. – using the resources available with the contractor. Accordingly the drawings made were very elaborate and in detail, inviting very few explanatory queries from contractors during construction, except where changes were needed due to site anomalies. The amount of work put in by our draughting team can be gauged by the number of drawings and sketches prepared – 600 A1 sheets, just by two draughtsmen in 20 months ! Limited use of STRUCAD was also made to explain with clarity some complex structural connection details.

DURING CONSTRUCTION

Our site quality monitoring team did an excellent job by taking it upon themselves to bring various aspects of design intents and associated quality standards expected to the attention of the contractor's men in the field and in the workshop. A systematic checklist was developed to ensure sequential fabrication and erection procedure for all elements. Periodic inspections by our team were useful in identifying deviations from the intended quality standards and in rectifying them on the spot, or expediting the process by direct and instant communication from site to the design office.

About the Authors:

Girish Dravid is Director of
Sterling Engineering Consultancy Services Pvt. Ltd.
E=mail : gdravid@bom3.vsnl.net.in

PIPE LINE ROLLING ACROSS PILE BENTS FOR THE TATA POWER CO LTD TROMBAY

A.B. Karnik

1 Introduction

This is very interesting project about installation of the COOLING WATER PIPELINE. Pipe line of large diameter carrying sea water to the THERMAL POWER STATION was to be installed on the top of pile bents. It was a big challenge to launch this pipe line over the pile bents having 30 m centre to centre distance between them.

2. Alternative Solutions

One way of execution was to place the 30 m long pieces of pipe in its position and then weld them at junction to make it continuous for the total length of pipe line. But this system has drawbacks. Firstly, it was difficult to carry such long (30m) pipes to the site from shore by ships. Also lifting the pipes from floating barge and then place it had limitations. Secondly, on site welding of pipe pieces can impose the possibility of leakage at the junction of pipe. Pipe of such a large diameter would be subjected to huge live load due to flowing fluid in it. So it was not desirable.

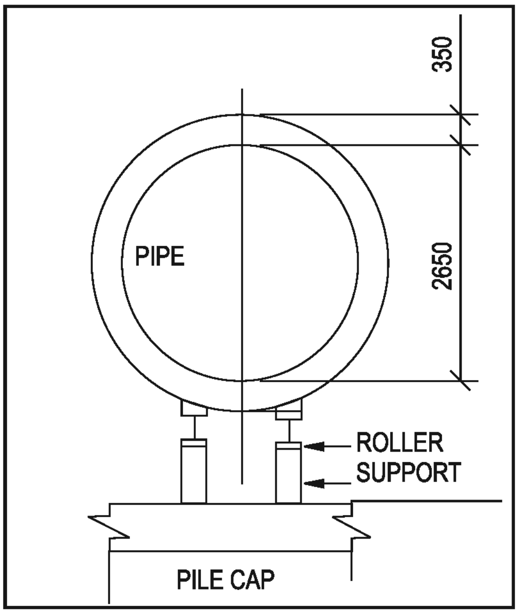


Figure 2. Scheme of Pipe and Supporting Girder

3. Adopted Scheme of Launching (refer sketches)

It was necessary to launch the pipe line continuously over all the spans. Hence, pipe was thought to be pushed from one end of the pipe line. Interestingly, the pipe was continuously fabricated at rear end. It facilitated continuous launching of pipe. It also avoided the delay due to carrying of pipes to the site and erecting them. Hence, it was most economical and comfortable solution for erection.

Pipe was already designed for its working loads and live load of water. The large span of 30 m would lead to large amount of bending moments and shear forces on the pipe bearing the live load. Pipe was provided with the stiffener rings, at every interval of 2.5 m to avoid buckling all along its length. Also the thickness of pipe was increased for the length of 5 m at support to resist large shear at support. Two stiffener plates PL350X20 at 400mm apart were provided at support.

The concern of this article is design of launching system of pipe for the erection stage. In normal practice an erection girder, rolling on the supports, is designed to carry the pipe and take all the loads at erection stage. But here, the author utilized strength of Pipe itself to act as a girder to move on the roller supports. The higher moment of Inertia of pipe was utilized and the cost of fabrication of erection girder was saved. It again made the design very economical.

To roll this pipe, it was necessary to design the supporting beams B1 and B2 (refer figure 1&2) having common bottom surface to roll on the rollers fixed at support (as shown in the figure 10, 11 & 12). Thus, at the interval of stiffener rings, stiffeners plates are welded to connect pipe with the beams B1 or B2.

4. Loading Considered (refer figure 6)

Let's first calculate the loads acting on the girder.

4.1 UDL due to Self Weight of Pipe + 12 thick Plate + Stiffener Rings at 2500 mm c/c

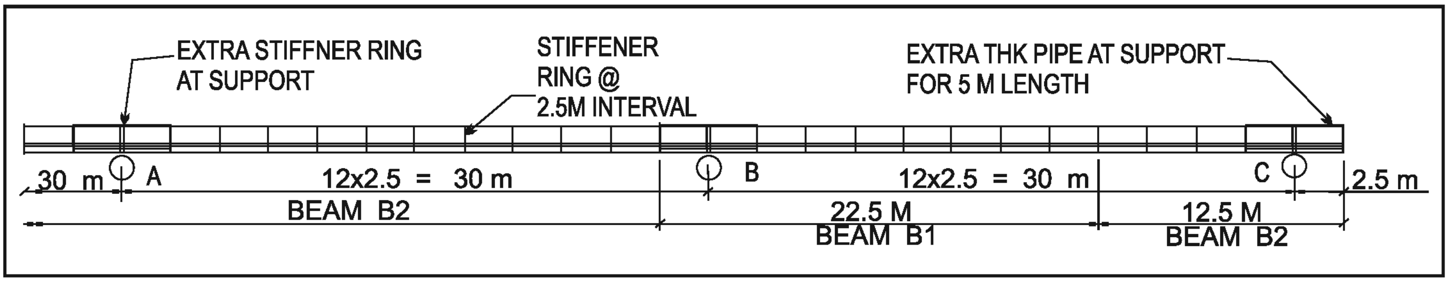


Figure 1. Typical Pipe Line Launching for TATA POWER CO. LTD. TROMBAY

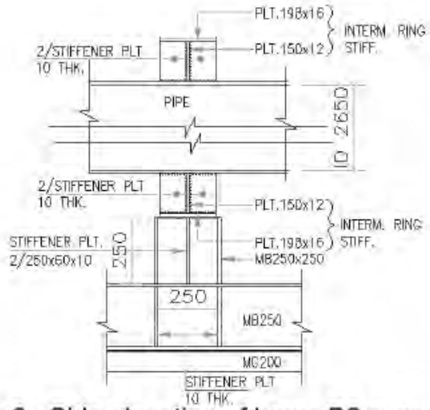


Figure 3 . Side elevation of beam B2 supporting the pipe and its stiffener T-ring

4.1.1 UDL due to self Wt of 12mm thick pipe with inner dia of 2.65 m is 788 kg/m

4.1.2 Add the wt of T ring provided (as shown in figure 3) composed of PL150X12 and PL 198X16 at interval 2.5 m

Wt of T ring at interval of 2.5m was 360 kg.

Hence, Wt of stiffening system per m = 360/2.5 = 144 kg/m

W1 = wt of pipe + wt of stiffeners = 788 + 144 = 932 kg/m = 0.932 t/m ≈ 1 t/m

4.2. Loading on the Girder at Support

4.2.1. Loading at support due to 4 mm extra thickness of pipe for 5 m at support.

UDL for 2.5 m on either sides of support due to Wt of extra thickness of pipe, W2 is 264 kg/ m

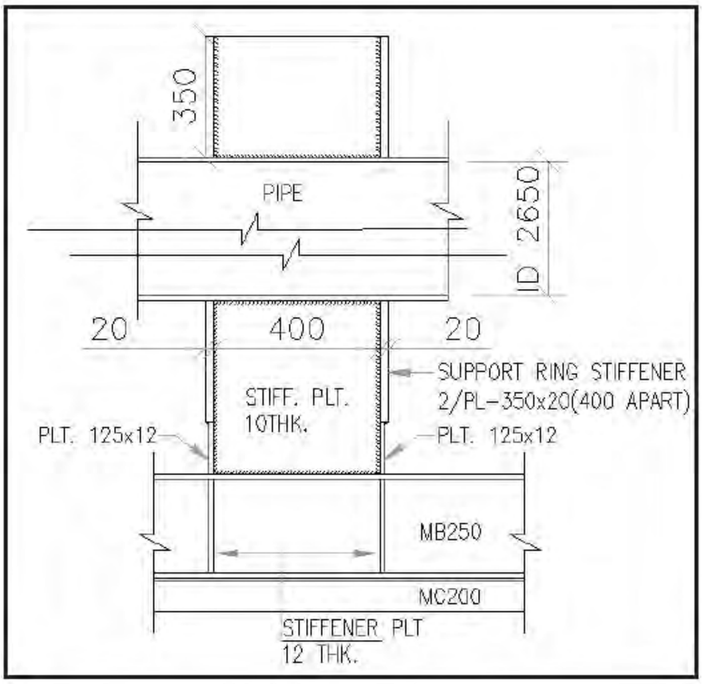


Figure 4. Side Elevation of Beam B2 Supporting Pile at Support

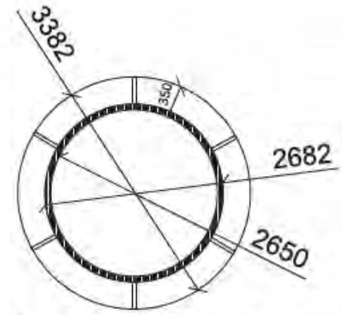


Figure 5. Cross Section of Stiffener System at Support

Equivalent point load due to wt of extra thickness for 5 m of span = 264 X 5 = 1321 kg.

4.3 Load due to Stiffener at Support

Provide 2 stiffener plates PL.350 X 20, 400 mm apart (refer figure 4&5)

4.3.1 Outer dia of stiffener ring = dia of pipe + 2 x width of ring = 2.674 + 0.350 X 2 = 3.374m

Wt of 2 stiffener rings over support = 2X π/4 x (3.374^2 - 2.674^2) x 0.02 x 7850 = 1044 kg

4.3.2 Provide 6 No of PL.400 x12, between stiffener rings 400 mm apart

Wt of 6 No of PL.400 x 12 = 6 x 0.4 x 0.350 x 0.012x 7850 = 79 kg

Wt of stiffener at support, W3 = 1044 + 79 = 1123 ≈ 1125 kg

4.4 Hence, consider the UDL of 1t/m over entire length and total point load at support due to W2 & W3 =

1321 + 1125 = 2446 kg ≈2.45 ton

5. Analysis

First, let us find out the support reactions for different spanning conditions. These reactions will be taken by two rollers (ref figure 10, 11) . Considering the reactions and bending moments at critical condition of max cantilever, max deflection of the pipe acting as girder, will be calculated.

5.1 To calculate load on rollers

Consider the critical conditions of support

5.1.1 Pipe is 2.5 m cantilever beyond point C

Negative Bending moment will be maximum at support, when the cantilever length will be maximum or full span cantilever; that is 30 m. As the pipe will be considered to be supported at 30 m only when, the stiffener ring designed to be at support, will touch the support C. Hence, push the pipe 2.5 m ahead of point C. This is the situation just before the pipe gets supported at support C. Hence this is considered as 30 + 2.5 = 32.5 m of cantilever condition.

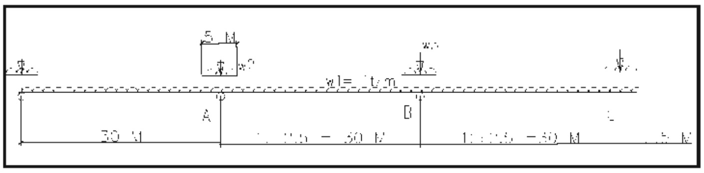


Figure 6: Loading on beam when pipe is 2.5m cantilever beyond point C

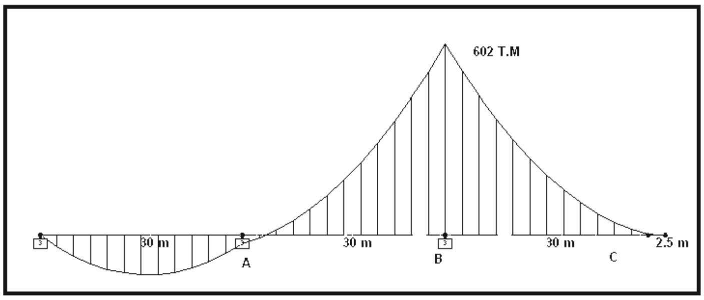


Figure 7. Bending Moment Diagram when Pipe is 2.5 m Cantilever beyond point C (case 5.1.1)

Taking moment about point A

$M_A = 1 \times 62.5^2/2 + 2.45(60+30) - 30R_B = 0$

Hence Reaction at support B,

$R_B = 1/30\ (1 \times 62.5^2/2 + 2.45(60+30)) = 72.45$ T

Max moment at support B,

$M_B = 1 \times 32.5^2/2 + 2.45 \times 30 = 602$ t.m (ref figure 7)

Here, two preceding spans are considered for analysis. It is considered that in the continuous spans of similar lengths, the effect of moment due to cantilever dies down approximately after 2 spans.

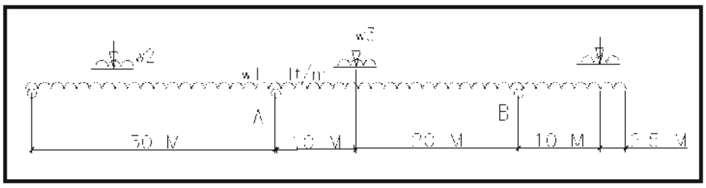


Figure 8. Loading diagram when cantilever end is 12.5 m to the right of support B (Case 5.1.2)

5.1.2: Critical condition for Penultimate Span when Cantilever end is 12.5 m to the Right of Support B

Reaction at support B,

$R_B = 1/30\ (1 \times 42.5^2/2 + 2.45(40+10)) = 34.19$ T

5.1.3 For all Continuous Spans of 30 m

Reaction at support A, R_A for typical spans

= 1X30 +2.45 = 32.45 T

5.1.4 **Reaction will be shared by Two Rollers at One Support.**

For 51.1, load on rollers = 72.45/2 = 36.23 T

For 5.1.2, load on rollers = 34.19/2 = 17.10 T

For 5.1.3, load on rollers = 32.45/2 = 16.23 T

5.2 To calculate Rotation at B and Deflection at Cantilever end

Rotation and deflection of the girder are inversely proportional to Moment of Inertia of Girder. Greater the MI, lesser is the values of rotation θ & deflection ∂. Hence the idea of using pipe itself to act as a girder is very beneficial here to get minimum deflection. . Also, the contribution of beam B1 or B2 will be very small than that of Pipe. Hence neglect the properties of B1 and B2, and consider that of only pipe.

Even though the thickness of pipe is increased at support, thickness of pipe is considered 12 mm for overall length to calculate rotation and deflection

MI of 12 thick pipe (O.D. = 2.674m, I.D. = 2.65m, Average Dia. = 2.662 m)

$I_{pipe} = \pi/64 \times (267.4^4 - 265^4) = 8.89 \times 10^6\ cm^4$

Section modulus $Z_{pipe} = 8.89 \times 10^6/\ (2.662/2) = 0.6649 \times 10^5$ cm^3

Bending stress fbc = $M/\ Z_{pipe} = 602 \times 10^5/0.6649 \times 10^5 =$ 905 kg/cm^2

Rotation at support B = $\theta_B = 3.39 \times 10^{10}/EI$

Maximum Deflection at cantilever end ∂ =15.6 cm

6. Design of Beams Supporting the Pipe

Two beams (Type B1 or B2) 1000 mm c/c will support pipe longitudinally (ref figure 10, 11& 12) and move forward on rollers. The reactions calculated in 5.1 are transferred to 2 rollers through these beams.

Load is transferred from pipe to Beam at the interval of 2.5m through stiffener rings as point load (refer figure 9). Overall bending of the girder is resisted by pipe. Hence the beams B1 and B2 will be designed only for roller reactions and local bending of beam when the support is in between the two stiffener rings. Consider the same maximum reactions, calculated in 5.1, at mid of span 2.5m, for design of beams B1 and B2. (Refer figure 9)

First length of 12.5 m beam is designed as B2 for the load of 17.1 T (case 5.1.2).

The reaction will go on increasing till cantilever length increases from 12.5 m to 32.5 m. Hence this portion of 32.5 -12.5 = 20 m will be designed as beam type B1 for load of 36.23 T (case 5.1.1)

Once the cantilever length of 32.5 m becomes simply supported in B and C, support reaction is reduced. Also rest all the supports will have the reaction as case 5.1.3 which is almost equal to that of 5.1.2.

Hence, Rest of the length of beams is also designed as Beam type B2

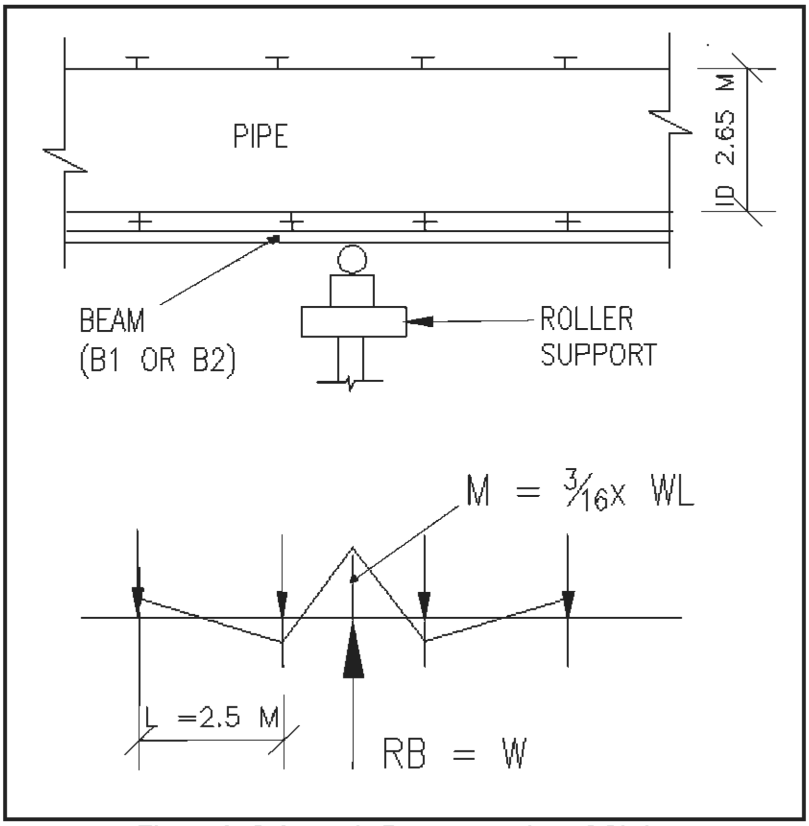


Figure 9. Schematic Representation of Girder and support for Local bending of beam B1 or B2

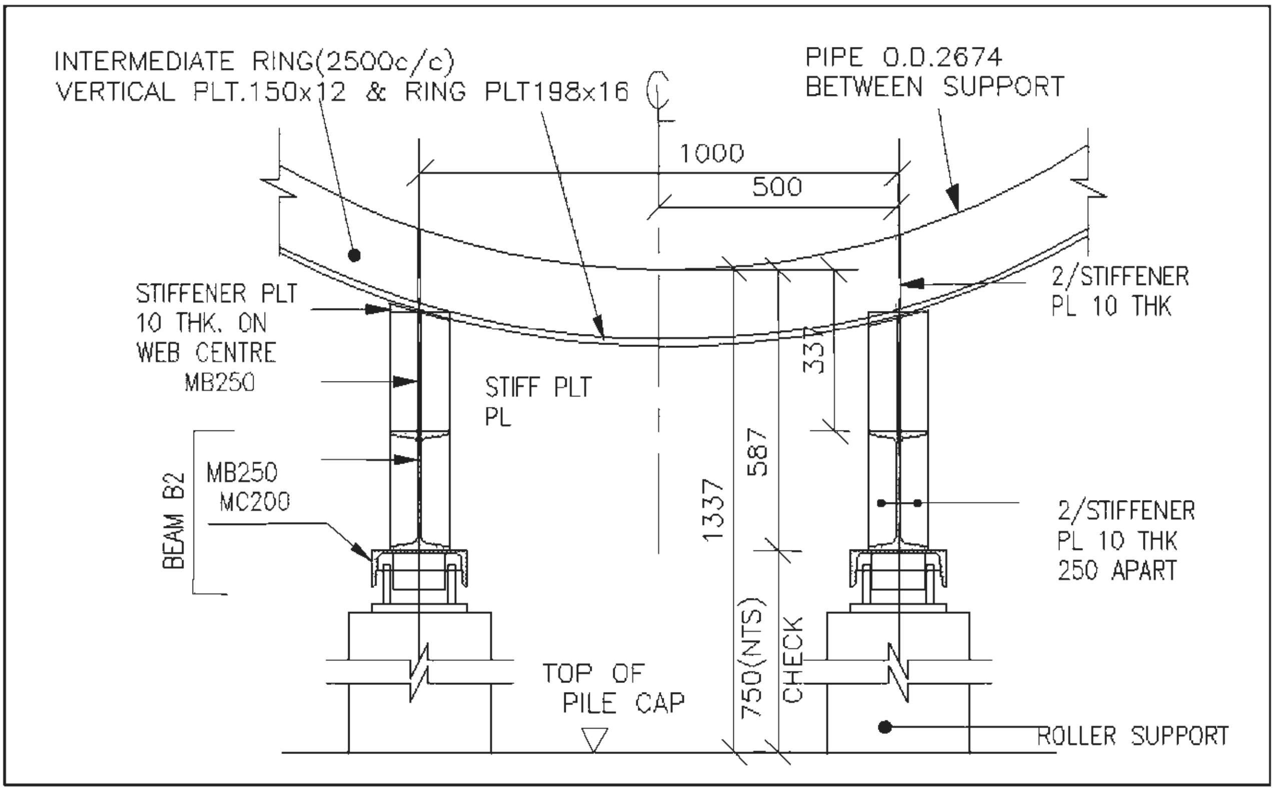


Figure 10. Details of Pipe Attachment for Rolling over Pile Caps (Intermediate T-Rings at 2.5 m c/c)

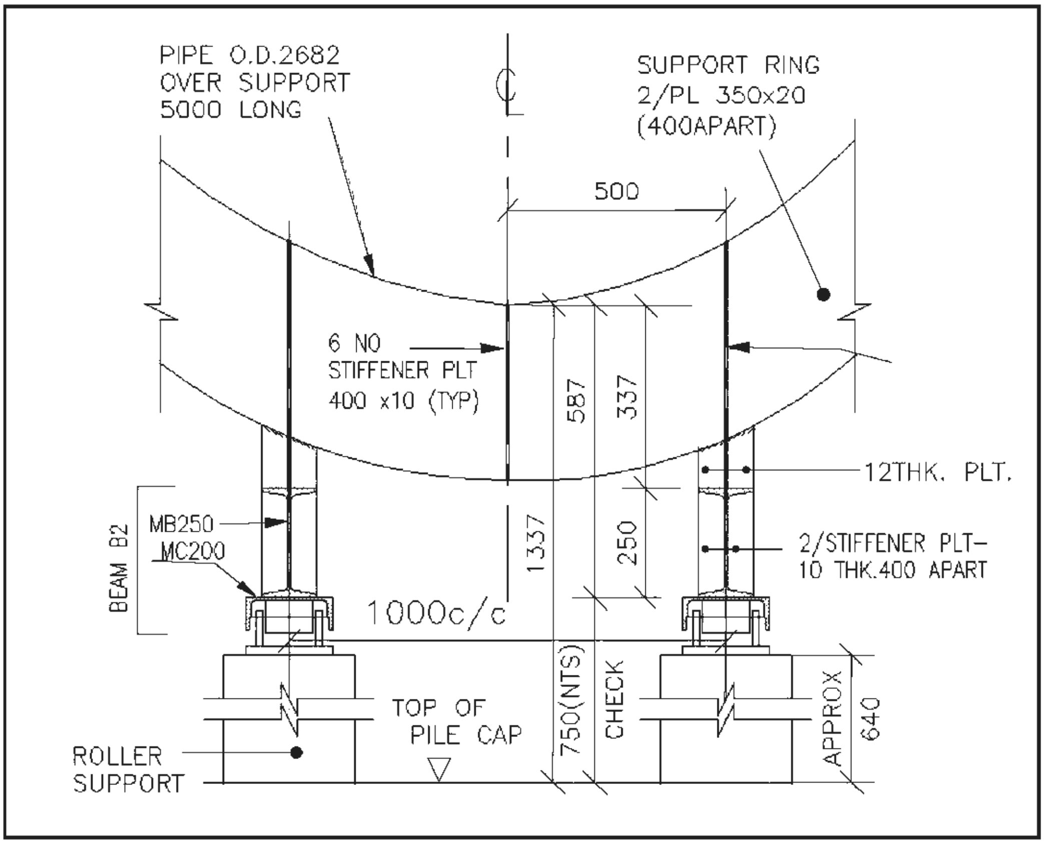


Figure 11. Details of Pipe Attachment for Rolling over Pile Caps (At support)

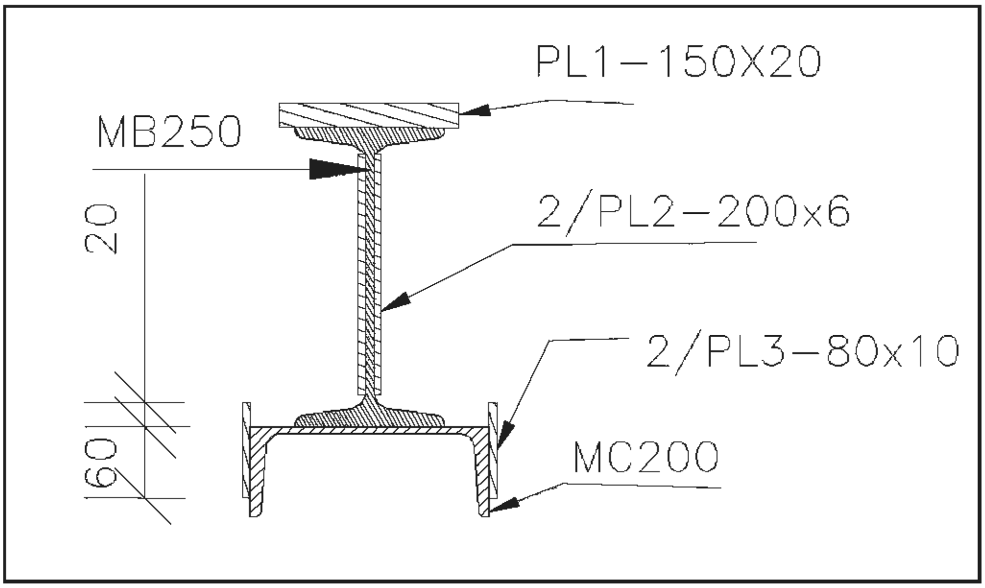


Figure 12. Section of B1 (Add plates PL1, PL2 & PL3 to B2)

Properties of Sections Used

MB250 c/s Area = 47.55 cm^2, I_x = 5131.6 cm4

MC200 c/s area = 28.21 cm^2 , I_{yy} = 140 cm4

Area of PL1 – 150x20 =30 cm^2

Area of PL2 2/200X6 =24cm^2

Area of PL3 2/80x10 = 16cm^2

Calculations for beam B1 and B2 are Tabulated further.

Table for Stresses in Beam B1 and Beam B2

Data	unit	Beam B1 (ref figure 12)	Beam B2 (ref figure 10, 11)
Shear force, S	T	36.23	17.1
Bending Moment, M	T.m	(3/16) X 36.23 X 2.5 = 17 Case 5.1.1	= (3/16) X17.1 X 2.5 = 8 Case 5.1.2
Define Sections Used for Beams			
		MB250 + MC200 + PL PL1, PL2, Pl3	MB250 + Mc200
Total Depth Of Section	cm	2 + 25 +7.5 = 34.5	25 + 7.5 = 32.5
Distance Of Neutral Axis From Top, Y_t	cm	16.36	17.96
Distance Of Neutral Axis From Bottom, Y_b	cm	18.14	14.54
Moment of Inertia of Combined Section $I = \Sigma AY^2$	cm^4	19783	9082.4
Section Modulus At Top $Z_{top} = I/Y_t$	cm^3	1209	505.7
Section modulus at bottom $Z_{bot} = I/Y_b$	cm^3	1091	624.62
Shear Stresses In Beam fs= Shear Force/area resisting shear(or web area)	Kg/cm^2	36230/(25X.69 +24) = 878	17100/(25 x 0 69) = 991
Permissible shear stress in beam	Kg/cm^2	1000 > 878 Hence OK	= 1000 > 991 Hence OK
Bending Stresses In Compression $F_{bcomp} = M/Z_{bot}$	Kg/cm^2	$17x10^5/1091$ = 1558	$8X10^5/624.62$ = 1280.77
Permissible bending stress in compression	Kg/cm^2	= 1650 > 1558 Hence OK	= 1650 > 1280.77 Hence OK
Bending Stresses In Tension $F_{btension} = M/Z_{top}$	Kg/cm^2	$17x10^5/1209 = 1406$	$8x10^5/505.7 = 1581.96$
Permissible bending stress in tension	Kg/cm^2	= 1650 > 1406 Hence OK	= 1650 > 1581.96 Hence OK

Final erection of pipe :

Utilizing the properties of pipe, it was launched by itself. Once, all the length of the pipe was placed at its final position, the pipe with the stiffener system was lifted with the help of Hydraulic Jacks. The supporting beams B1 and B2 and roller support system for rolling the pipe was removed. Finally, pipe can rest on the pile cap and act as continuous beams. Thus this project could be successfully completed in record time with the innovative solution.

About the author :

A.B. Karnik
Senior Consulting Civil Eng.
E-mail : arunodaya22@gmail.com

EARTHQUAKE RESISTANT DESIGN OF STRUCURE & EXPERIMENAL ANALYSIS OF SCALED MODEL

Samir Dhuri

*I happen to be the external examiner for the final year student's project work at VJTI on the above subject. It was a pleasant surprise for me to scrutinize the outstanding work done by a team of undergraduate students at VJTI. I congratulate the team for their hard work , enthusiasm and their successful participation in the competition conducted by EERI, US. I hope ISSE members will find this work interesting and inspiring . - **Editor (Hemant Vadalkar)***

1. Introduction :

We are a team of undergraduate students from Veermata Jijabai Technological Institute (VJTI), Mumbai. Our team members are driven to make a mark in the field of earthquake sustainability. As a part of this, we students participated in Seismic Design Competition 2012 which is organized by EERI (Earthquake Engineering Research Institute). We wish to share this experience with ISSE. We were the 1st team from India to participate in this competition. With this discussion, more undergraduate students will develop interest in this field.

The objectives of competition were to promote study of earthquake engineering among undergraduate students. It gave us opportunity to work on a hands-on project designing and constructing a cost-effective frame building to resist seismic loading and understand the fundamentals of Structural Dynamics and Earthquake. Our team was hired to submit a design for a multi-story commercial office building. Our task was to design and construct a cost-effective structure to resist seismic loading.

2. The overall framework of competition is as follows:

To verify the seismic load resistance system, a scaled balsa wood model must be constructed and tested. The model is to be subjected to three ground motions, which represent different return period earthquakes. In order to ensure life safety the building model must not collapse during shaking. In addition, the response of the model in terms of roof drift and roof acceleration will be measured during the shaking. For each ground motion, the value of the roof drift will be used to estimate the monetary loss due to damage in the structural and non-structural building components. Likewise, the roof acceleration will be used to estimate the monetary loss due to damaged equipment that is contained inside the building. If collapse occurs, the monetary losses will account for demolishment, reconstruction, and downtime. Finally, the annual seismic cost will be obtained as the sum of the economic loss estimated for each of the earthquakes divided by its return period. A cost-benefit analysis will be carried out to determine the most cost-effective building. This will be done by balancing the revenue with the initial building cost and seismic cost.

Project Constraints:

The model has to comply with the conditions provided by the organizers as follows:

2.1 Building Dimensions

The building must comply with the following dimensions.
Max floor plan dimension: 15 in x15 in
Min individual floor dimension: 6 in x 6 in
Max and min number of floor levels: 29 and 15
Floor height: 2 in,
Lobby level height (1st level): 4 in
Min and max building height: 32 in and 60 in
Max rentable total floor area: 4650 in2 (3 m^2)

2.2 Weight of Scale Model

The total weight of the scale model, including the base and roof plates and any damping devices, should not exceed 4.85 lbs (2.2 kg). The approximate weight of Model Base Plate and Roof Plate 1.1 Kilograms.

2.3 Structural Frame Members

Structures shall be made of balsa wood. The maximum member cross section dimensions are:
Rectangular member: 1/4 in x 1/4 in and Circular member of dia 1/4 in (6.4 mm).

2.4 Shear Walls

Shear walls constructed out of balsa wood must comply with the following requirements:
Maximum thickness: 1/8 in, Minimum length (plan view): 1 in.
Shear walls must span at least one floor. Structural members can attach to the ends of a shear wall.

2.5 Structural Loading

Dead loads and inertial masses will be added through steel threaded bars tightened with washers and nuts. These will be firmly attached to the frame in the direction perpendicular to shaking.
Floor weight: (1.18 kg)
Roof weight: (1.59 kg)
Weight spacing: Increments of 1/10th the height (H/10)
Length between weights on bar: 16 in Threaded bar diameter: 1/2 in
The dead load will be placed at nine floor levels in increments of (H/10), corresponding to (1/10) x H to (9/10) x H. In cases where a floor does not exist at an exact increment of (H/10), the weight will be attached to the nearest higher floor.

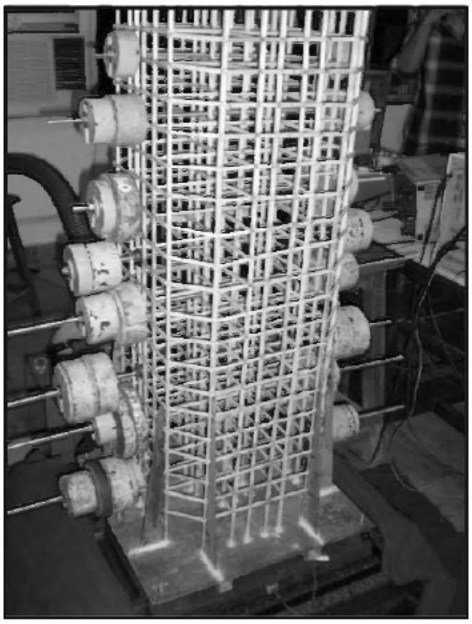


Figure 1. Testing of model on shake table with the loads applied

3. Our aim

To achieve balance of a stiff and flexible structure in design with Simple structural configuration. The design must be Cost effective and feasible.

4. We proceeded for our project as follows:

Before we could start working on actual model, various tests were performed. Following points were considered first:

4.1 TESTING

4.1.1Material

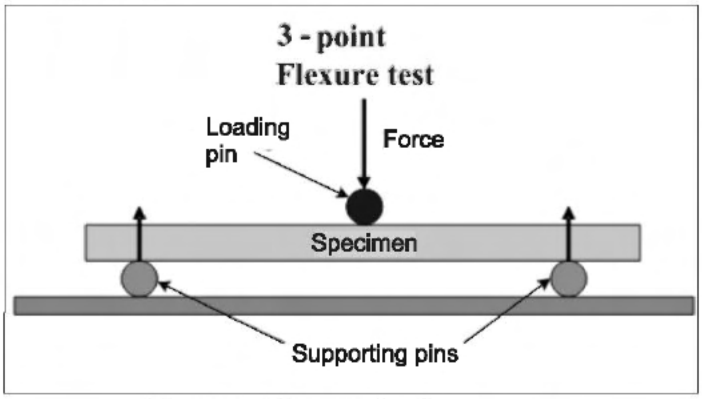


Figure 2. Three point flexure test

4.1.1.1 BALSA wood:

The material which is very close to steel is BALSA wood, which is very delicate to handle, as well as tough to work on. As it is brittle, it is liable to fracture and slicing and hence incapable to take considerable strain and deformations. Three point flexure test was performed (refer Figure 2) to determine Elastic modulus of balsa wood from formula

$E = (5wl^4)/384Iy$.

4.1.1.2 Adhesives:

Table 1. Strength of Adhesives

Adhesive	Tensile strength	Shear Strength
Fevikwik-	9.5 kg/mm^2	0.4 kg/mm^2
Araldite	4.8 kg/mm^2	15.8 kg/mm^2
Fevicol	3.1 kg/mm^2/	11.08 kg/mm^2

Figure 3. Gusset plate connection in model

4.1.2 Connection

No connection was supposed to be made with nails or binding wires etc. Only method to connect to members was to use

Groove Joint, Butt Joint, Tongue & Groove Joint, and Oblique Cross Halving Joint to enhance the strength of joint. Also, Gusset Plate is used for bracing connection; it increased surface area of contact and joint fixity.

4.1.3 Compression testing of column

A prototype of beams and column of 2 storeys is subjected to compression test and the model with desired results is adopted.(refer Figure 4)

Figure 4. Compression test of prototype (buckling of columns at middle beam joints observed)

4.2 EXPERIMENTS:

To achieve our final model, we made 3 models before.

First, the scheme of framework of model was tested in ETABS software. Model with the optimum solution was chosen to be erected. The model is then tested on the shake table to study the effect of vibration on the structure simulating to the earthquake force.

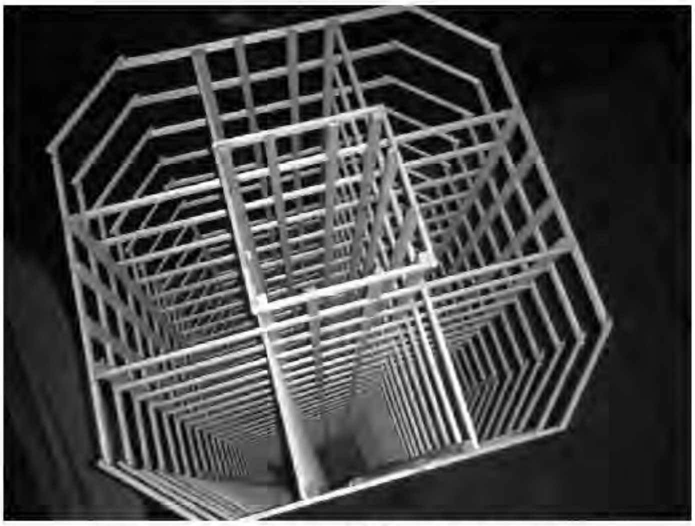


Figure 5: Primary models without bracing

First model was made of pine wood due to unavailability of balsa wood in India and then actual balsa wood was used for rest of the models. Tube-In-Tube form was used for model 1 and 2. No bracing were provided to understand the behavior of the columns and beam frame structure only. Also, the method of erection was improvised from experience. In first model, no more than 3 people could work on the model. But later, it was erected in parts and the structure was made horizontal while assembly. So proper level is achieved.

4.3 Conceptual input in final model:

From the experiments, we modified our structure to achieve final model. The features are:

4.3.1 Stiffen the structure:

Structure becomes stiffer if the internal force is more direct, more uniformly distributed and small. This concept is explained with the equations:

Deflection of pin jointed structure is = $\Sigma\ (N_i^2 L_i \,/\, E_i A_i)$ For i= 1 to m

Stiffness of pin jointed structure is = $1/\Sigma\ (N_i^2 L_i \,/\, E_i A_i) = 1/\ \Sigma N_i^2 f_i$

Where,

N_i: is the i^{th} internal force induced by a unit load at the critical point;

L_i: is the length of i^{th} member;

E_i: Young's modulus of the i^{th} member;

A_i: area of the i^{th} member;

$f_i = L_i/E_i{*}A_i$ is known as the flexibility of the i^{th} member.

Hence to increase the stiffness conceptual solutions used are

1. As many force components as possible should be zero.
2. No one force component should be significantly larger than the other non-zero forces.
3. The values of all non-zero force components should be as small as possible.

4.3.2 Core area was increased compared to primary models to increase the stiffness.

4.3.3 Shear Walls:

We tried Tapered shear walls which cause reduction in cost, requirement of lesser material, faster completion of project. But it led to formation of soft-storey mechanism due to gradual reduction of shear resisting area, and the structure failed in torsion. Hence, we decided to use continuous shear wall for full height of the structure.

4.3.4 Super Column

General super column theory consists of provision of columns having large dimensions and concrete columns having steel sections.

They are extended to the top of the structure and provide a high degree of rigidity and generally adopted for tall structure. This was used to design huge right angled triangle shaped hollow columns. First, It was made using three shear walls converging to form right-angled triangles at the four corners of the structure. In later stage, only inclined side was made of shear wall and other two mutually perpendicular sides were formed by braced system of columns to reduce weight.

This concept of super column helped to concentrate max weight along the edges and hence use the weight of the structure with maximum benefit.

This gave very good performance during software modeling in spite of absence of a core and provision of outriggers.

Also, it provided an overall stiffness and symmetry to the structure. This design causes lower amount of torsion. Also very large spans could be provided which is always welcomed by the developers.

4.3.5 Bracing

Bracing systems was used for transmitting loads and increasing lateral structural stiffness. Bracing members should be provided in each story from the support (base) to the top of the structure. Bracing members in different stories should be directly linked so as to ensure proper transmission of loads and to improve performance. Bracing members should be linked linearly wherever possible. For a temporary grandstand structure, however, the number of bays is usually larger than the number of the stories. Bracing members in the top story and in different bays should be directly linked where possible. To create a shorter force path, or more zero force members in a structure, consider the relationship of bracing members across the bays of the structure. If extra bracing members are required, usually to reduce bracing member forces and distributes them more uniformly, they should be arranged as stated above.

4.3.6. Roof frame:

As the roof frame was subjected to loading, the frame of roof was made of thicker section to avoid local failure.
(Refer Figure 6)
]

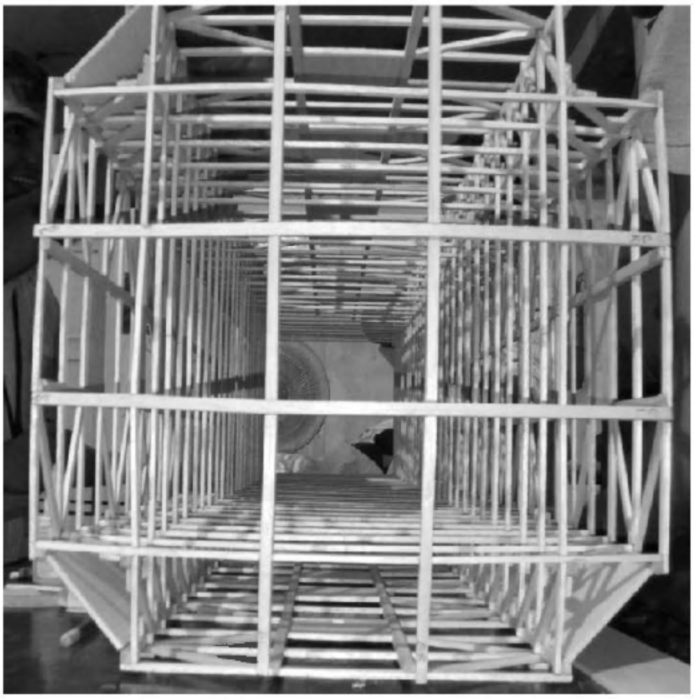


Figure 6: Thicker members in Roof grid

4.3.7 ETABS

To decide what kind of bracing configuration to be used on the final model various models were made based upon the above concepts in E-tabs. The models were so prepared that other than bracing configuration nothing was changed. These models were analysed for a given time history function and based upon the deflection the final pattern of bracing configuration was decided.

Characteristic of testing were
Period: 1sec
Number of steps per cycle: 20
Number of cycle: 5
Amplitude: 0.101 g

We tested several designs in ETABS and decide to construct more better structure every time. (Refer figure 8 and tabel 2)
The effect of construction errors can significantly hamper the performance of building during earthquakes thus emphasizing the importance of quality assurance.
The performance prediction of scaled building models were exactly matching with software modelling due to accuate

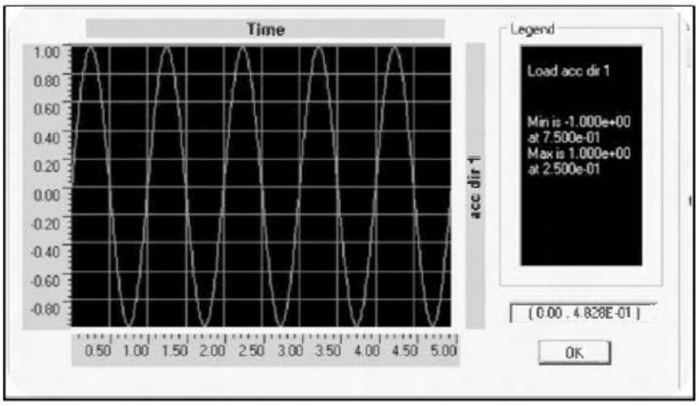


Figure 7. FFT analyzer to convert the time domain response to frequency domain

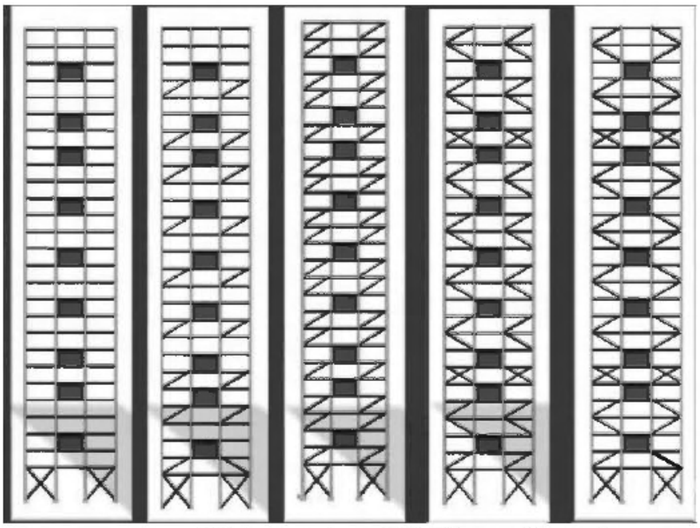


Figure 8: Five Trial models in ETABS for different types of bracings

Table 2. Model testing results using ETABS software

Model	Displacement mm	Natural period sec	Natural frequency Hz
Trial one	0.5738mm	0.127641	7.8344
Trial two	0.5408mm	0.123654	8.0870
Trial three	0.5166mm	0.120926	8.2695
Trial four	0.4900mm	0.117978	8.4761
Trial five	0.4891mm	0.117896	8.4820

assumptions of material properties, joint fixity and geometry and precise construction. There was considerabel difference in actual model (refer figure 9) and software consideration of centre to centre joint. We need to make some assumptions and changes in input for ETABS model.Say, connection doesn't offer total rigidity to the joint, in ETAB model, we reduced the value of fixity of joint in the software model hence achieving similarity between software model and the actual model.

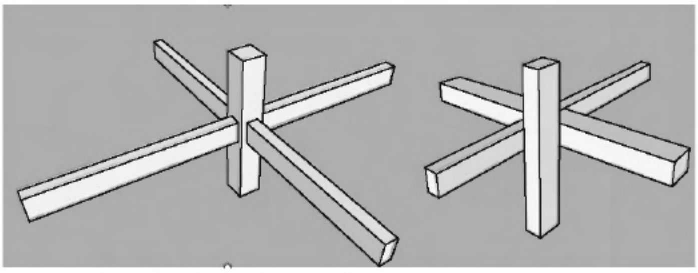


Figure 9: Difference of connection in software and in actual model

4.4 Final testing

In the final testing, the joint of structure to the base plate failed. The reason being, we practiced using M seal to connect the structure to the base plate, which transmits the vibration to the model. But the same was ineffective due to increased setting time in the lower temperature then and there.

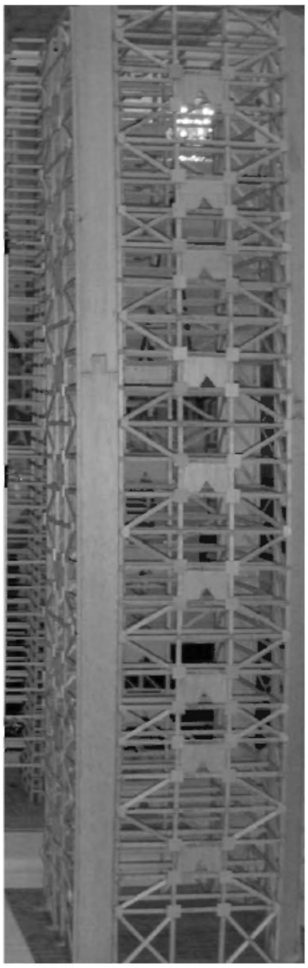


Figure 10. Final model

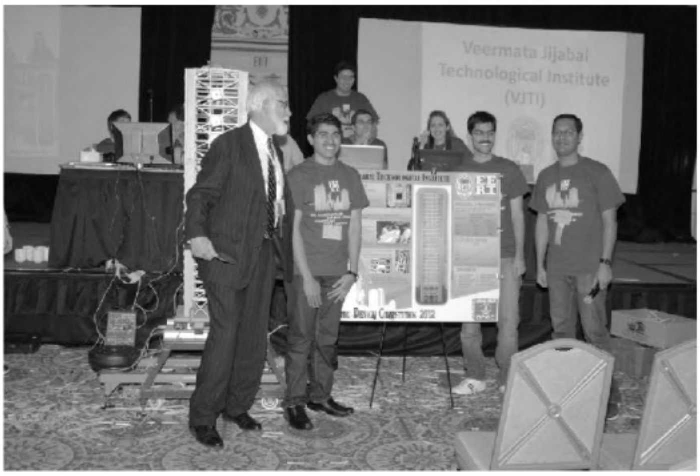


Figure 11. Testing of our model at EERI (VJTI student with the president of EERI)

5. Conclusions

This project has imparted us a great exposure to the field of structral engineering and its parts like structural dynamics, experimental mechanics, structural analysis to earthquake engineering, finite element analysis, geotechnical engineeering. Every field has its equal importance and in together they help us to make megastructures stand every natural calamities like earthquake, storm etc.

We made it to US and landed at the 18th position out of 34 universities & 6th amongst the Broken Structures category. We were the only team to have exact roof Drift & acceleration prediction in EERI SDC 2012.

References

1. "Improving the Earthquake Resitance of Small Buildings, Houses and Community Infrastructure".- Gregory A. J. Szakats 12th WCEE.
2. FEMA 451-B Concepts of Earthquake Engineering
3. Practical Three Dimensional Nonlinear Static Pushover – Ashraf Habibullah, and Stephen Pyle- ETABS.
4. Dynamic Analysis using Mode Superposition- CSI
5. IBC Internatioal Building Code-2009
6. NEHRP -Code 7(National Earhquake Hazard Risk Prevention)

7. "Earthquake Tips" – C. V. R Murthy-NICEE (National information Center of Earthquake Engineering)

8. "The Design Integration of Structural Efficiency, Architectural Expressiin and High Performance Exterior Wall Sytems".- Mark Sarkisian CTBUH(Conference of Tall Buildings and Urban Habitat

9. "Seismo Resisting Architecture on Building Scale"- H Guliani , M I Yacante, A M Campora.

10. "Design Example of a Six Storeyed Building"- Sudhir K Jain

About the author:

Sameer Dhuri, is student of final year B. Tech. VJTI and group leader. Group Secretary of EERI-VJTI Student Chapter. He can be reached at samsdhuri@gmail.com

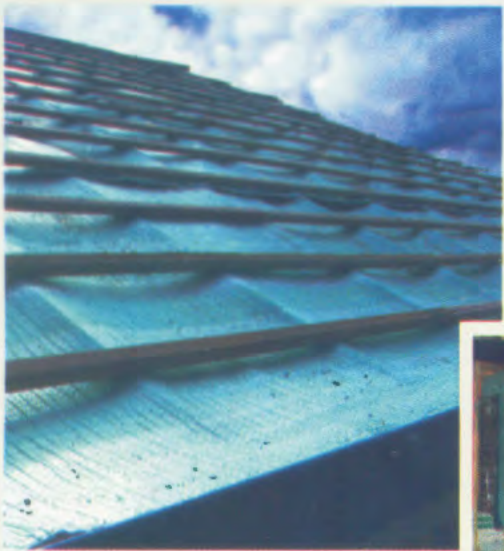
For more details visit:
http://slc.eeri.org/SDC2012.htm ,
http://www.youtube.com/watch?v=MtfTBGrZMiw
http://www.vjtiunshakeables.com/

Edited and published by N K Bhattacharyya for ISSE, C/o S G Dharmadhikari, 24 , Pandit Niwas, 3rd floor S K Bole Marg, Dadar, Mumbai 400028. Tel91-22-24365240, Fax-9122-24224096, email 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

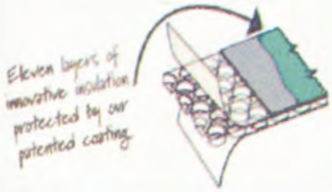
Green Insulation
THE INTELLIGENT SOLUTION

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