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CONTENTS

Editorial

Message from President, IStructE Mr. Manoj Mittal

From the Desk of Chief Editor Dr. S. Chatterjee

Technical Articles

Procedure, Principles and Practices in Tall Buildings Mr. Hemant Gor

IS 16700- Criteria for Structural Safety of Tall Concrete Buildings Ms. Alpa Sheth

Easy to Execute Seismic Upgrade and Retrofit of a Steel Hospital Building Using Fluid Viscous Dampers Dr. Amir Gilani Dr. Kit Miyamoto Mr. Sandeep Shah

Role of Codes & Standards in Structural Engineering Mr. Alok Bhowmick

Waste-Derived Manufactured Aggregate for Concrete Permitted in IS: 383 – 2016 Dr. Ajoy Mullick

Shiva Statue at Nathdwara Rajasthan Dr. Abhay Gupta

Bridge Foundations in Strata with Potential of Liquefaction Prof. Mahesh Tandon

Operational Model Analysis of a Large Truss Bridge: A Case Study Dr. Nirmalendu Debnath Dr. Anjan Dutta Dr. Sajal Kanti Deb

Challenges Faced (Then and Now) during Kolkata Metro Construction – A Study Prof. Mainak Ghosal

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Dear Readers,

It gives me a great pleasure to announce that IAStructE is resuming the official publication of the technical journal "Structural Engineering Digest (SED)". I am sure the contents of the journal will be very useful to the professional structural/civil engineers.

It will also provide a platform for reporting the latest technical achievements/advancement as well as the new tools and techniques available to the structural engineers. Academics and researchers would be approached to contribute in the journal to expose the readers with what is the latest and what is coming in this field, thus, to help enhancing the knowledge base and to improve quality of service. I am happy to let you know that editorial team headed by Dr. S Chatterjee has done an excellent job in bringing out the first issue of SED in its new format. It is intended to be a truly technical journal of structural engineering.

While wishing you all good reading of SED, I solicit and would very much appreciate your contribution in any form towards the success of SED.

Best regards,

Manoj Mittal
President
The quarterly technical journal of Indian Association of Structural Engineers, Structural Engineering Digest (SED), made its debut in the initial years of the association and got established as a good technical journal for the structural engineers, quickly. It continued to appear almost regularly for a number of years. However, the journey could not be sustained and somewhere it lost its stream, went into oblivion. There must have been multiple reasons for that to happen but it would serve no purpose to dwell on the why and how, in this forum. There has been an urge to revive this useful journal from time to time. Finally, a few months back, it was decided to make a restart immediately, with the first issue to come out in July, 2018. The mission is not only to revive the journal, but also to further improve its focus and quality, so that it becomes a reference journal for the structural engineers, from the industry, as well as, from the academia, in the long run.

It was also decided that some of the issues should be theme based, addressing specific areas of contemporary importance. The target set was to bring out the first issue in July, 2018. However, various logistics issues have delayed the publication by a few weeks. This is also not a theme specific issue. We hope to be able to bring out the subsequent issues regularly, on time, which will require cooperation from all the readers, by contributing good quality technical articles and in getting advertisements towards, making it financially viable. I hope you will receive this issue enthusiastically and extend your help in improving the quality for the future issues.

We have been able to attract a few of the leaders in structural engineering to grace our editorial team. We could not obtain their full involvement for this issue, due to constraints of time and availability. These are teething problems, not expected to last. It has been a difficult journey to make this issue see the light of the day. This could be achieved because everyone has been supportive, particularly the authors, the members of the editorial board and most profoundly, Mr. Manoj Mittal, President, and Mr. Alok Bhowmik, Secretary. My sincere thanks to all of them and to Mr. Vikas Verma, Manager, for his constant support from the Secretariat.

Kind regards.

S. Chatterjee
Chief Editor
Abstract:
This paper integrates information on process, principles and practices followed in design and execution of Tall Buildings. This paper describes various lateral load resisting system used in Tall building and how to manage biggest enemy of tall building-Wind loading. Tall building projects are outcome of collaborative approach among multi-discipline expertise and problem-solving attitude.

Introduction
Tall building project brings together high strength materials, experienced professionals and innovations. Burj Khalifa which signifies human ambition to rise higher reflects an integration of these three critical aspects:
- High strength concrete till 601m and 227m steel spire at top
- Collaboration among Architect, Structural Engineer, Wind Engineer, Building Facility Engineers, Façade Engineer, Geotech Engineer and Construction
- Innovation in concrete technology, lateral load resisting system and elevator

This paper presents following key aspects of tall building projects:
- Design Stages
- Lateral Load Resisting Systems
- Strategies to manage wind load
- Design challenges

Design Stages
Design of Tall buildings have following three stages
1. Conceptual Design (Competition Stage) (CD)
2. Preliminary Design (PD)
3. Detailed Design (DD)
**Conceptual Design (CD)**

At this Stage Structural Engineer closely works with Architect and Client representative to understand objective of the tall building. This stage structural engineer works on a broad concept to transfer loads safely to the foundation, constructability and economics. Figure 1 presents disciplines working together on challenges, alternative viable solutions and inter-discipline impact. The key decisions freeze at this stage are

1. Orientation and External form of Building to ensure minimum wind loads
2. Floor System (Material of construction and arrangement)
3. Lateral load resisting system
4. Constructability
5. Approximate cost of construction

Structural engineer presents two alternative viable solutions along with sketches to describe load path and construction sequence. Structural engineer collaborates with Architect, Geotechnical engineer, Wind Engineering Firms, MEP (Mechanical, Electrical and Piping) and Construction experts to capture all critical input in the conceptual design. At this stage, structural engineer considers the tall building as a single giant cantilever beam to arrive stiffness requirement for lateral load resisting system (Refer Figure 2).

**Preliminary Design (PD)**

The second stage of the tall building project is preliminary design. The key objective of this stage is to generate contract drawings and quantity estimate for tendering process.

At this stage, detail 3D Computer analysis model is generated in leading software ETABS, SAP2000.

Geotech investigation agency is appointed to carry out soil exploration at the site. Geotech consultant recommends the type of foundation, allowable bearing capacity and stiffness for Soil-Structure interaction model.

Wind Consultant is appointed to perform wind tunnel study and recommend modifications for sustainable design.

A typical wind-tunnel test to evaluate structural loading on a building consists of the following steps:

1. Simulate the natural wind environment in the tunnel, including profiles of mean speed and turbulence, including both the far field (ambient approach conditions) and near field (the localized effects of nearby buildings or topography).
2. Construct a model geometrically scaled to the building, place it in the simulated environment, and “observe” what happens using appropriate instrumentation.
3. Obtain the initial dynamic characteristics of the building from the structural engineer.
4. Define the site wind climatology and assign design wind speeds.
5. Analyze and interpret the observed results, in view of design speeds and structural dynamic properties, to...
obtain static-equivalent loads and accelerations.

(6) Interact with the structural engineer to refine or optimize the structure

The test provides wind load at each floor level for along wind load, across wind loading and torsional moment. Following three tests

<table>
<thead>
<tr>
<th>Table 1: Wind Tunnel Test Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>High Frequency Force Balance Test</strong></td>
</tr>
<tr>
<td>Purpose</td>
</tr>
<tr>
<td>Duration</td>
</tr>
<tr>
<td>Input</td>
</tr>
</tbody>
</table>

**Detailed Design (DD)**

This is the third and final stage of Tall Building Engineering. During this stage working/execution drawing and supporting calculations are produced for PEER review. 3D Computer analysis model is refined to reflect final geometry, stiffness, construction sequence and loadings. Key elements are analysed and designed to transfer loads between elements. BIM (Building Information Modelling) model is generated to help the progress of construction, design clash detection and visualization. For seismic load, Performance Based Design performed to ensure structural integrity during an extreme event. CONTRACTOR provides a detailed method statement, technology to execute work safely, Construction Plan and Quality Assurance/Quality Control Plan. CONSULTANT provides a schedule of engineering drawings and documents for the site.

**Aspect Ratio**

The overall slenderness of a tall building is usually defined by its aspect ratio also called “Height to base ratio" being the height of the building divided by its narrowest plan dimension (refer Figure 3). Preferably the value of aspect ratio is less than 7. Aspect ratio of world's second tallest building "Shanghai Tower" with height 632m is 7. Tall buildings with aspect ratio above 8

![Slenderness Ratio, SR = h/b]

Where,  
- **h** = building height  
- **w** = building width  
- **b** = building breadth  
and where  
- **b < w**

![Figure 3: Aspect ratio of tall Building (Reference 1)]

**Lateral Load Resisting System**

The lateral load resisting structure (LLRS) -also referred to as lateral stability system consists of all structural elements which form part of the load path(s) for transmitting the lateral effects of all loads (wind, earthquake, eccentric gravity effects, unbalanced lateral earth pressures load) from their sources to the foundation. These elements typically include walls, vertical truss, beams, columns and floor diaphragms.

The effects of wind on LLRS can be considered in two parts, namely:

(a) ULS load effects affecting the strength design of LLRS;
(b) wind motions due to dynamic response to SLS wind loads affecting the serviceability of the structure in terms of human perception (peak acceleration felt at top floors).

The general principle behind the efficient design of LLRS of tall buildings is to engage the perimeter structure (gravity columns) with the core(s) within the constraints of planning and architecture. By effectively engaging the perimeter gravity columns the structural width of the LLRS (and hence its efficiency) is increased dramatically. Also as the perimeter columns are preloaded in compression due to gravity loads, they can resist wind-induced tensions (or, more accurately, decompression) very economically (with minimum need for tensile reinforcements). Adjacent cores are often engaged together with "header beams" (typically running across the lobbies) allowing the core boxes to act in a compound manner in resisting the lateral loads, i.e., to develop "push-pull" couple over their cross-sections rather than simply bending independently. The perimeter columns can be engaged with the core(s) by either direct or indirect "shear linkage" elements. The direct shear links can be provided by outrigger walls connecting the core(s) and the columns. The indirect shear links can be provided by offset outriggers, belt walls, and the like

Table 1 presents lateral load resisting system development from simple moment frame to present Buttressed core.
Table 2: Lateral Load Resisting System and Representative Building Example

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Name of Building</th>
<th>Height of Building (m)</th>
<th>No of floors (Above/Below Ground)</th>
<th>Year of finish</th>
<th>Material of Construction</th>
<th>Slenderness (Height to Width Ratio)</th>
<th>Gross Floor Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Framed Tube</td>
<td>Aon Center</td>
<td>346</td>
<td>83 / 5</td>
<td>1973</td>
<td>Steel</td>
<td>5.85:1</td>
<td>334,448</td>
</tr>
<tr>
<td>Bundled Tube</td>
<td>Willis Tower (Sears Tower)</td>
<td>442.1</td>
<td>108 / 3</td>
<td>1974</td>
<td>Steel</td>
<td>7.45:1</td>
<td>416,000</td>
</tr>
<tr>
<td>Tube in Tube</td>
<td>432 Park Avenue</td>
<td>425</td>
<td>85</td>
<td>2015</td>
<td>Concrete</td>
<td>15:1</td>
<td>65,497</td>
</tr>
<tr>
<td>Trussed Tube</td>
<td>John Hancock Center</td>
<td>344</td>
<td>100</td>
<td>1969</td>
<td>Steel, Composite floor</td>
<td>6.84:1</td>
<td>260,126</td>
</tr>
<tr>
<td>Core + Outrigger</td>
<td>Taipei 101</td>
<td>508</td>
<td>101/5</td>
<td>2004</td>
<td>Composite Concrete</td>
<td>6.8:1</td>
<td>198,347</td>
</tr>
<tr>
<td>Buttressed Tube</td>
<td>Burj Khalifa</td>
<td>828m</td>
<td>163/1</td>
<td>2010</td>
<td>Concrete</td>
<td></td>
<td>309,473</td>
</tr>
</tbody>
</table>

Figure 4 – Aon Center, Framed Tube – Closely spaced perimeter columns tied together with beams

Figure 5 – Sears tower, Bundled Tube – Closely spaced perimeter columns tied together with beams
Strategies to manage Wind

The tall building design is governed by wind loading. Following strategies are used to manage wind loading in tall building

**Table 3: Managing Wind load**

<table>
<thead>
<tr>
<th>Strategy</th>
<th>Example Building</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner cut and corner recession (5 to 10% Building width)</td>
<td>Taipei 101</td>
</tr>
<tr>
<td>Tapering along height</td>
<td>Petronas Tower</td>
</tr>
<tr>
<td>Setbacks along height</td>
<td>Sears Tower</td>
</tr>
<tr>
<td>Orientation</td>
<td>Burj Khalifa (Nose oriented along predominant wind direction)</td>
</tr>
<tr>
<td>Twisting</td>
<td>Shanghai Tower</td>
</tr>
<tr>
<td>Corner chamfered</td>
<td>World Trade Center (Twin Towers)</td>
</tr>
<tr>
<td>Elliptical shape in plan</td>
<td>Al Mas Tower, Dubai Guangzhou International Finance Center, China</td>
</tr>
</tbody>
</table>
In Dubai prevailing extreme wind direction is North – West. This can be observed in Google map for Dubai Airport runway orientation (See Figure 10-b). Figure 10-a shows direction of lower impact wind direction which is along one of the nose. Both this information was integrated to orient tower (refer figure 10-b) resulting in most economical design.

**Concrete for Tall Buildings**

Traditionally, compressive strength tests are performed at 28 days, but many high-rise buildings now requiring high strength concrete (HSC) employ a construction schedule whereby the structural elements in the lower floors are not fully loaded for periods of a year or more. Under these circumstances, it is reasonable to specify compressive strengths based on either 56- or 90-day results, thereby taking advantage of the strength gain that occurs after 28 days. For Burj Khalifa compressive strength of wall at the lower floor was specified Cube compressive strength 80MPa and Young's modulus 43,800MPa at 90 Days.

Concrete provides following advantages in the tall building:

- The mass and rigidity of concrete provides twice the dampening effect compared to steel, reducing forces on super-tall buildings due to wind and the cost of construction
- Structural concrete is naturally fire resistant and sound insulation
- Advancements in concrete pumping technology, including the introduction of placing booms, make easy, fast delivery of concrete possible, freeing tower cranes for other work.
- Improvements in concrete mixes, including strength and modulus of elasticity (E) have made high-rise construction more attractive. Self-consolidating concrete (SCC) is increasing in use too.
Approximate concrete consumed in tall building can be arrived using formula recommended by Dar's formula (4)

Volume of Concrete in m$^3$/m$^2$ Gross Floor Area (GFA) = 
\[(0.62N+15) \text{ m}^3/\text{m}^2 \text{ to } (0.62N+60) \text{ m}^3/\text{m}^2\]

For 80 Storied Tower with total Gross floor area 130,000m$^2$, Concrete Volume will be in range of
\[V_{\text{max}}=(0.62 \times 80 + 60) \times 130,000 = 14.248 \text{ Million m}^3\]
\[V_{\text{min}}=(0.62 \times 80 + 15) \times 130,000 = 8.398 \text{ Million m}^3\]

**Conclusion**

The pillar for completion of the tall building project is integrated collaboration among multi-discipline. Tall building Design involves broadly three stages, Concept Design, Preliminary Design and Detail Design. An idealized 2D beam model of the building will give sufficient results for the preliminary design. However for final detailing BD analysis should be carried out. Increase in height of building provides an opportunity to innovate new Lateral Load Resisting System (LLRS). The LLRS which is most viable for 300m tall tower does not suffice for the 600m tall tower. Using various strategies to manage wind load on tall building may reduce overturning moment at the base by around 25% on tall buildings giving overall cost saving. Inherent mass, damping, durability and fire resistance of structural concrete makes it most suitable material for the Tall Buildings.

**References**

1. Tall Buildings, A working group of the Concrete Centre and fib Task Group 1.6
5. Wind Tunnel Methods, D Boggs and A Lepage, ACI SP 240-6
9. Tall Building Initiative, Guidelines for Performance Based Seismic Design of Tall Buildings, PEER, April 2017
10. Tall Buildings, Concrete Quarterly Special Issue No 1, The Concrete Centre
11. Case history Report, the world's tallest concrete skyscrapers. Bulletin No 40, CRSI
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IS 16700- Criteria for Structural Safety of Tall Concrete Buildings

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Managing Director, VMS Consultants Pvt. Ltd. vmsb@vakilmehhtasheth.com

Introduction

As India experiences rapid development, cities will continue to see a huge spurt in the demand for affordable housing and commercial real estate, not just in the metro cities but also in tier 2, 3 and 4 cities. In response to this need, the urban development ministries of the states have increased the allowable built up area on land by means of augmenting Floor Area Ratio (FAR or FSI). Most cities now typically have new buildings of 15 storeys and higher (50m+) to consume the available FAR. The structural engineering community across the country was not geared to the sudden increase in building heights and there were gaps in the conceptualisation and design process of tall buildings. Unlike low-rise buildings, design of taller buildings are driven not by gravity loads alone; lateral loads such as wind and earthquake play a defining role in conceptualising the design.

A standardized design protocol to ensure acceptable performance of tall structures in terms of safety and serviceability was needed. Such a Standard Code of Practice did not exist in India for design of tall buildings. Hence a new Standard for Criteria for Structural Safety of Tall Concrete Buildings was developed.

This standard provides prescriptive requirements for design of reinforced concrete tall buildings. The following salient aspects, which are based on the prescriptive approach, are addressed in this standard:

a) Structural systems that can be adopted in tall building;

b) General requirements including: 1) height limitations of different structural systems, 2) elevation and plan aspect ratios, 3) lateral drift, 4) storey stiffness and strength, 5) density of buildings, 6) modes of vibration, 7) floor systems, 8) materials, and 9) progressive collapse mechanism;

c) Wind and seismic effects: 1) load combinations, and 2) acceptable serviceability criteria for lateral accelerations;

d) Methods of structural analysis to be adopted, and section properties (in cracked and uncracked states) of reinforced concrete member to be considered in analysis;
e) Structural design aspects for various applicable structural systems;
f) Issues to be considered in design of foundations; and

g) Systems needed for structural health monitoring.

As another first in the country, this code acknowledges that there will be buildings that will not conform to the requirements of the code and there should be a special review process for such "code-exceeding" buildings. For such buildings, the code has recommended detailed guidelines that may be adopted by local authorities which includes formation of a Review Committee and qualifications of constituent members for such a review committee.

Description

2.1 Rationale behind Prescriptive Nature of Code

As can be seen in Figure 1, almost 67% of the slum population in urban India lives in Tier 2 to Tier 4 cities. Under the revised Pradhan Mantri Awas Yojana (Urban) PMAY schemes, a significant section of slum-dwellers are envisaged to be rehoused in formal housing. Much of this affordable housing will be of over 50 m height due to constraints of land. The available technical resources for design and construction of high-rises in secondary tier areas is perceived to be limited. The reasons are two-fold. There is limited competent technical manpower available in the country. Secondly, the structural engineering community in India is poorly remunerated and as a result, suffers from a paucity of resources available at the disposal of the structural engineer to adequately invest in a rigorous design process required for performance based design approach. Hence, a prescriptive approach to design of tall buildings was considered more suitable to cater to the needs of the country.

Many iconic or other buildings, however, may not desire to follow the restrictive constraints imposed by the prescriptive approach. Such projects also offer greater resources to the designer to invest in a more rigorous analysis and design approach. For such buildings, the new code has suggested a detailed approval process in the Annexure A which is based on Performance-Based Design. Thus the code addresses the two extremes of the spectrum, Prescriptive Design, Performance-based Design, and everything in between.

Another reason for a prescriptive approach is that almost all structural design codes in India are prescriptive. Both the main earthquake code (IS 1893:2017) and the wind Code (IS 875:Part 3: 2015) are also prescriptive. While there is discussion that future revisions of the earthquake code may lean towards displaced-based design approach, this may take a while to be effected. It was felt prudent, under the circumstances, that the new Concrete Tall Buildings Code (IS 16700) follows, rather than leads, the march towards performance based design for all structures, so that the various design codes in India are more or less in conformance in their philosophy and approach.

Figure 1. Urban Population in India

As can be seen in Figure 1, almost 67% of the slum population in urban India lives in Tier 2 to Tier 4 cities. Under the revised Pradhan Mantri Awas Yojana (Urban) PMAY schemes, a significant section of slum-dwellers are envisaged to be rehoused in formal housing. Much of this affordable housing will be of over 50 m height due to constraints of land. The available technical resources for design and construction of high-rises in secondary tier areas is perceived to be limited. The reasons are two-fold. There is limited competent technical manpower available in the country. Secondly, the structural engineering community in India is poorly remunerated and as a result, suffers from a paucity of resources available at the disposal of the structural engineer to adequately invest in a rigorous design process required for performance based design approach. Hence, a prescriptive approach to design of tall buildings was considered more suitable to cater to the needs of the country.

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Figure 2. Displacement based Design Approach-Way of Future? Desirable Collapse Mechanism, and quantifiable Deformation Capacity

2.2 Height Elevation And Plan Aspect Ratios Limitations Of Different Structural Systems

The code restricts maximum height of a building for structural systems based on the seismic zone of the site. These restrictions have been historically imposed by other international codes such as ASCE 7 and Chinese Code JGJ-3 Concrete Structures of Tall Buildings and find mention in numerous research journals.
It has been generally found that an aspect ratio \( H/b \) (height versus least plan dimension) exceeding 10 results in very slender buildings that may require some damping device to reduce heightened perception of motion (akin to sea or carsickness). IS 16700 has thus restricted aspect ratio to 10. It may be pertinent to mention that there has been in recent times, a trend, however limited, of super-tall buildings with very high aspect ratio, also called "Pencil Towers". This trend is prevalent in New York and in recent times in cities such as Boston and Melbourne. None of the above mentioned cities have a severe or very severe seismic hazard. Los Angeles (LA) and San Francisco (SF) which have very severe seismic hazard do not have such pencil towers. Buildings in LA and SF that do not follow the prescriptive codes of ASCE 7, ACI 318 and other codes mandated by the local authority are required to follow special, non-prescriptive procedures as mandated by Los Angeles Tall Buildings Structural Design Council (LATBSDC) "An Alternative Procedure For Seismic Analysis And Design Of Tall Buildings Located In The Los Angeles Region" and by SEAONC, 2007, "Recommended Administrative Bulletin on the Seismic Design & Review of Tall Buildings Using Non-Prescriptive Procedures" respectively. The requirements are complex and need a detailed performance based design using non-linear analysis. As noted by F Naeim in his paper "The performance-based alternative procedure requires an in-depth understanding of ground shaking hazards, structural materials behavior, and nonlinear dynamic structural response. In particular, the implementation of this procedure requires proficiency in structural and earthquake engineering including knowledge of: seismic hazard analysis and the selection and scaling of ground motions; nonlinear dynamic behavior of structural and foundation systems; mathematical modeling capable of reliable prediction of nonlinear behavior; capacity design principles; and detailing of elements to resist cyclic inelastic demands, and assessment of element strength, deformation and deterioration under cyclic inelastic loading."

2.3 Building Modes Under Seismic Loads

The IS 16700 code does not allow for torsion irregularity. It stipulates that the first two modes of vibration should be translational modes and the torsion mode cannot be earlier than the third mode of vibration with a mode spacing of at least 10% between consecutive modes. This requirement is in line with the revised IS 1893 code definition of torsional and vertical irregularities. This requirement creates a challenge for buildings symmetrical across both principal axes where the two
lateral modes of vibration may be less than 10% apart (even though torsional mode is third mode of vibration) and some variation in stiffness in the two directions would be required to be introduced to avoid this lateral storey (Vertical) irregularity.

2.4 Use of Precast and Historical Experience of its Seismic Performance in the Country

The prescriptive approach of IS 16700 does not cover use of precast elements in tall buildings. It further does not allow use of precast floor systems without a concrete topping of 75 mm in seismic zones III to V. The performance of precast structures in the 2001 Bhuj earthquake was very poor. A project of building new precast schools, executed from April 1999 to November 2000 and just before the Jan 26 2001 earthquake, proved to be disastrous. About three-fourths of these newly built precast schools either collapsed or were seriously damaged.

<table>
<thead>
<tr>
<th>Type of Damage</th>
<th>Number</th>
<th>% of Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A</td>
<td>48</td>
<td>15.0</td>
</tr>
<tr>
<td>Type B</td>
<td>90</td>
<td>28.2</td>
</tr>
<tr>
<td>Type C</td>
<td>140</td>
<td>43.9</td>
</tr>
<tr>
<td>Type D</td>
<td>30</td>
<td>9.4</td>
</tr>
<tr>
<td>No Damage</td>
<td>11</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Figure 4. Statistics of damage to precast school buildings in the Kachchh region in 2001 Bhuj Earthquake (See Figure 5 for damage types).

Type A damage — major damage to structure.

Type B damage — roof planks slipped.

Type C damage — minor dislocation of roof planks.

Type D damage — minor opening of grouted joints.

Figure 5. Damage Types A to D in 2001 Bhuj Earthquake.
There has not been adequate documented research in detailing and testing of precast joints subsequent to their catastrophic behaviour in 2001 earthquake as a result of which there is still some scepticism in the academic and research community to allow for unchecked use of precast elements in buildings. Future research, development and rigorous testing in this field may pave the way for review of codal provisions on this subject.

2.4 Openings in Diaphragms

Floors and floor systems including roofs are the key players in distribution of lateral loads (seismic, wind) through their "diaphragm" behaviour, performing as deep beams with very high in-plane stiffness and strength in comparison. They are typically modelled as "rigid" diaphragms i.e. they have infinite in-plane stiffness. It is thus assumed that there is no in-plane deformation in the floor plate. For such an assumption to be valid, restrictions are required to be imposed on the floor openings as large cut-outs and cut-outs along periphery in the plan of the building do not allow for rigid in-plane behaviour and this results in uneven distribution of the inertia force mobilized at floor levels during earthquake shaking. This distribution and irregular behaviour cannot be captured by simplified "prescriptive" approach to design. Diaphragm opening restrictions are imposed in most international codes using prescriptive approach. Figure 6 displays the results of a study of effect of % openings in diaphragm to incorrectness of rigid diaphragm assumption (measured as ratio of maximum displacement of diaphragm to the average displacement of diaphragm) based on studies conducted by Murty et al.

2.5 Limits on Concrete Grade

In view of the brittle behaviour of certain types of High Strength concrete, the code has imposed caution in use of concrete grades beyond M70, while not completely disallowing them. Annexure B gives some direction on use of high strength concrete.

2.6 Progressive Collapse Mechanism and Key Elements

The code makes a passing reference to the issue of progressive collapse mechanism and key elements, suggesting the need for redundancy through alternate load path. The intent is to draw the attention of the engineer to important issues of key elements and collapse mechanisms. However, as buildings with transfer systems etc. are outside the prescriptive approach of this code, the subject has not been delved in much detail and will fall within the purview of approval procedures for "code-exceeding buildings".

2.7 Wind Effects

The code has stipulated wind tunnel studies for buildings greater than 150 m in height and those with complexities of shape, topography or are very flexible (T_e >5s). The code requires the building to be designed for the wind tunnel results, subject to minimum of 70% of that derived from the wind code (IS 875: Part 3). By this provision, the code is giving the option to the structural engineer to design the structure utilising more realistic wind load and its distribution obtained from boundary...
layer wind tunnel studies. The lower bound of 70% is to cater for changes in the interference effects due to new developments in the vicinity of the building. Lateral Accelerations have been capped at 0.15% and 0.25% for residential and mercantile buildings respectively. There is paucity of credible data on building movements in existing structures in India. It is hoped that by recommending building monitoring systems, new data on building movements and occupant reception and comfort will emerge, which will inform code writers on this issue in future code revision.

2.7 Earthquake Effects

IS 16700 has made no significant departure in evaluating earthquake loads or in earthquake resistant design. However, the code has relaxed (reduced) the minimum design base shear coefficient to be used for buildings for 200 m in height by 30%. This relaxation is in keeping with the historical seismic data available that has consistently demonstrated the wide spacing between the frequency range of very tall buildings and the frequency range of earthquake shaking.

2.8 Structural Analysis

Use of cracked section properties in earthquake resistant design was initiated by the revised earthquake code IS 1893-Part 1 2016 which stipulated cracked section properties to be used for cracked reinforced concrete structures. IS 16700 has taken this a step further by stipulating separate set of cracked section properties to be used for elastic design (wind loading) in addition to cracked section properties for earthquake resistant design in line with IS 1893.

2.8.1 Backstay Effect

The code has a detailed section on backstay effects. The section acknowledges the typology of tall buildings in India and recognises that developers and clients are loathe to provide a seismic joint between the tower and the surrounding podium due to serviceability issues.

As mentioned in the code, "backstays" transfer lateral forces from the tower elements to elements of the podium or basement which are outside tower footprint. This transfer helps the tower in resisting overturning moments. As shown in figure 8a, this transfer of lateral forces due to additional support from podium/ basement floors can cause a reversal of shear forces.

As part of collapse prevention evaluation, the code requires two sets of backstay sensitivity analyses to be carried out using upper-bound and lower-bound cracked section properties of floor diaphragms. The stiffness parameters for those diaphragms and perimeter walls of podium and below the level of the backstay are given in Figure 9. These analyses are in addition to those required to be carried out using other cracked section properties.

<table>
<thead>
<tr>
<th>Sl No.</th>
<th>Stiffness Parameter</th>
<th>Values to Be Adopted</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Upper-Bound</td>
</tr>
<tr>
<td>(1)</td>
<td></td>
<td>(2)</td>
</tr>
<tr>
<td>(1)</td>
<td>$I_{eff}/I_{g}$</td>
<td>0.5</td>
</tr>
<tr>
<td>(2)</td>
<td>$A_{ct}/A_{g}$</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**Figure 9: Stiffness Parameters**

In estimating backstay effects, the code stipulates that two lateral load paths shall be considered, namely:

1) Direct load path, where overturning resistance is provided by the tower elements and foundation directly beneath the tower; and

2) Backstay load path, where overturning resistance provided by in-plane forces in the backstay elements (lower floor diaphragm and perimeter walls).
They code identifies the different cases when backstay effect will occur and requires the backstay floor diaphragms to be modeled considering their actual in-plane and out-of-plane stiffness shall be designed for the maximum of forces derived from sensitivity analysis.

For plan irregularity as per Table 5 of IS 1893 (Part 1) of Type i, Type ii and Type iv as well as Type iv of Table 6 (vertical irregularities) seismic forces shall be amplified by a factor of 1.5 in the design of connections of diaphragms to vertical elements and to collectors; and for design of collectors and their connections.

For irregularity of Type iii (diaphragm discontinuity), seismic forces to be amplified by an over-strength factor of 2.5.

Backstay diaphragms have to be designed for maximum forces obtained from sensitivity analysis and must be minimum 150 thick with two layers of steel in each direction and 0.25% reinforcement in each direction.

There are many more detailed requirements provided in the code for design of backstay elements.

### 2.8.2 Structural Walls

The code has revised minimum thickness of structural walls as 160 mm (200 mm in seismic zones IV and V) to align with fire code requirements (min 2 hours rating). It has further identified cut-out sizes in walls that may be ignored in analysis and design beyond which they will need to be modelled. Minimum reinforcement of 0.4% in each direction (in two layers) is to be provided.

### 2.9 Foundations

IS 16700 has made significant changes in design of Foundations for Tall Buildings. It has specified factor of safety of 1.5 for unfactored wind and gravity loads and 2.5 for design earthquake and unfactored gravity loads. Modelling raft foundations will require use of zoned spring constants or zoned modulus of sub-grade reaction for dead load + live load condition. For buildings taller than 150 m, a soil-structure interaction study to conducted

Further the allowable deformation of mats has been relaxed to 125 mm (50 mm in rock) subject to angular distortion being within 1/500. These limits are in keeping with international practice. The depth of foundation is required to be min 1/15 in raft and 1/20 for piled foundations to account for uplift issues. The requirement may be relaxed when there is not net uplift anywhere in the foundation.

The soil investigation requirements are stringent. Minimum 3 boreholes are required in each tower with depth of minimum 1.5 times width of foundation and 30 m in rock. While borehole investigation in rock of 30m may seem excessive, there are regions in India where rock is underlain by soft layers and this 30 m is meant to capture such situation.

### 2.10 Non-Structural Elements

IS 16700 has detailed design guidelines for securing of non-structural elements. The underpinning rational and design background is available in another document. The discussion of non-structural elements has its genesis in the fact that there
is more economic damage in earthquakes and wind conditions from damage to non-structural rather than structural elements.

It is pertinent to discuss the issue of liability herein. As structural engineers in India typically sign detailed contracts for structural design of buildings, and are not remunerated for non-structural element design and performance, the introduction of this topic in a structural code which will be binding to designers will require renegotiation of contracts or special waiver clauses in contracts.

2.11 Monitoring Systems

As mentioned earlier, there is paucity of credible data on building movements in existing structures in India. It is hoped that by recommending building monitoring systems, new data on building movements and occupant reception and comfort will emerge, which will inform code writers on this issue in future code revision. To this effect the code has recommended monitoring of instrumenting top of towers over 150 m height with tri-axial accelerometers for capturing earthquake shaking and anemometers and accelerometers for wind movement. Further, it has recommended monitoring of foundation settlements by means of settlement markers, pressure pads and strain gauges as relevant.

2.12 Annexures

As mentioned herein earlier, Annexure A provides detailed guidelines for setting up an approval process for "code-exceeding" buildings. The onus of forming the necessary review panel (ERP) rests on the local municipal bodies and the Bureau of Indian Standards, the code-making body in India which has published IS 16700 has little role to play in the implementation mechanism of the standards it creates. The setting up of these review panels is a formidable task and there will be many municipal bodies which may not have local resource persons qualified to be members of the panel. It would be recommended that such towns and cities use the services of larger neighbouring cities with such ERPs for buildings which are "code-exceeding" or delay the approval process for code-exceeding buildings until such committees are in place.

3.0 Concluding Remarks

For the first time in India, a special code for Design of Concrete Tall Buildings has been developed and was released in December 2017. This national code is unique in that other than China, no other country in the world has a separate and dedicated code for Design of Tall Buildings in concrete.

The code prescribes the minimum structural requirements of buildings so as to have a predictable structural behaviour and which allow the building to be designed by simpler, linear elastic (static and dynamic) procedures, eliminating the need for more complex dynamic non-linear analysis. When building configurations, structural systems and other parameters do not satisfy the code, the behaviour of such structures is not easily predictable and in such a case a more rigorous, non-linear analysis procedures would be required. The code also provided guidelines for the approval process for such buildings.

Many of the clauses in IS 16700 have raised concerns in the users. It may be argued, and not unfairly, that some of the clauses in the code are too restrictive, especially for high seismic zones. These issues are open to debate and discussion. Code writing is an ongoing process. A new code especially needs to go out into the world and be tested rigorously. Extensive use of this code and feedback will bring to the forefront the practical difficulties in implementing some of the provisions and will help accelerate the process of revising the code at an early date.

Reference


5. Murty CVR et al: "Introduction to Earthquake Protection of Non-Structural Elements", Gujarat State Disaster Management Authority 2013
Easy to Execute Seismic Upgrade and Retrofit of a Steel Hospital Building Using Fluid Viscous Dampers

(This article was also published in the Masterbuilder)

Amir Gilani, Kit Miyamoto and Sandeep Shah

ABSTRACT

India has never had a specialized seismic code for hospital buildings and, hence, in the event of an earthquake, it is expected that most if not all hospitals structures will have serious structural damage and many will even collapse. The pancake collapse of the Bhuj Hospital on 26 Jan 2001, wherein the patients and the hospital staff died after getting buried in the building debris, is still fresh in the minds of many. This paper presents a simple performance-based approach for upgrading existing multi-story hospital buildings. The procedures given are simple to design and easy to execute on site. The paper also specifies how the designer needs to select more stringent performance objectives in terms of flexure hinge demand than the minimum specified in the code/guidelines literature for important structures like a hospital building. The case study presented in this paper deals with a steel building, however the same principles can equally be applied to concrete buildings also. The same design methodology can also be used to design new steel buildings and will be significantly less expensive than if energy dissipation (dampers) were not used.

This paper presents a performance based approach to upgrade the seismic performance of a multi-story hospital structure located in Southern California. The uppermost eight stories of the superstructure use steel framed construction, whereas reinforced concrete framing is used for the lowest floor (L2) and the four sub-grade parking stories. The building is rectangular in plan, has a typical story height of 3.6 m (11.8 ft) and a total floor area of 12,000 m² (130,000 ft²). In both lateral directions, steel moment-resisting frames using Pre-Northridge connection...
details were used to resist lateral loading. Reinforced concrete framing comprise the lateral load resisting system for L2 and parking levels. Provisions of FEMA 273 [FEMA 1997], FEMA 351 [FEMA 2000], and OSHPD-approved design procedure [NYA&M 2003] were used to evaluate the expected seismic performance of the structure in the existing configuration subjected to a 475-year seismic event. A comprehensive three-dimensional mathematical model of the structure was prepared and subjected to acceleration histories generated to match the site-specific response spectrum. Response was evaluated using nonlinear analysis. Structural performance did not meet the design requirements for story drifts and nonlinear flexural rotations. To enhance the building response, the analytical model was modified by adding fluid viscous dampers attached to new exterior frames. Analysis of the structure in the upgraded configuration indicated that the performance requirements were satisfied. Parametric studies were conducted to investigate the effects of variation in key parameters, such as damper coefficients, nonlinear hinge properties and soil-structure interaction, on the response of the retrofitted structure.

INTRODUCTION

OVERVIEW This paper summarizes the analytical studies conducted to evaluate the seismic performance of a multi-story hospital building located in Southern California. For analysis and evaluation, provisions of the 2001 edition of California Building Code [CBC 2001] and FEMA 273 [FEMA 1997], and FEMA 351 [FEMA 2000] were utilized. Project-specific design guidelines [NYA & MI 2003] were constructed in accordance with the OSHPD’s specific requirements of the project. Analysis of the structure indicated that the story drifts and flexural hinge rotations exceeded FEMA 1997 limits and design guidelines. To mitigate these problems, fluid viscous dampers (FVDs) were sized and added to the analytical model. Nonlinear acceleration history analyses indicated that the upgraded structure complied with the design guideline limits for story drift and member nonlinear flexural rotations.

DESCRIPTION OF THE BUILDING The Hospital building is a nine-story building constructed over four levels of underground parking located in Southern California. The underground footprint exceeds that of the hospital building and supports an additional multi-story office structure. Only the seismic performance of the hospital building is addressed in this paper. The uppermost eight stories of the superstructure (levels L3 through roof) use steel framed construction, whereas reinforced concrete framing is used for the lowest story level (level L2). The building is rectangular and has plan dimensions of 21 m (70 ft) wide (in y- or NS direction) by 63 m (210 ft) long (in x- or EW direction). Figure 1 presents a photograph of the structure and typical floor plan.
resisting frame along all the perimeter and girder lines of the building. The steel moment frame connections in both the strong and weak-axis use Pre-Northridge (PN) details. The panel zones for the beam-to-column moment connections are unreinforced—no doubler plates were used. The lateral load resisting system for the first story level of the superstructure (L2) consists of a concrete moment-resisting frame along the perimeter and all girder lines of the floor. For columns, longitudinal reinforcement was either #36 (#11) or #44 (#14), and transverse reinforcement was #13 ties (#4) spaced at 100 mm (4 in.). For beams, top and bottom main longitudinal reinforcement was #32 (#10) bars. Beam main reinforcements were hooked into the column. The reinforcement had adequate development length and allowed beams and columns to develop their full moment capacity.

SEISMIC DEMAND The Design Basis Earthquake (DBE) was used for design and evaluation. GeoPentech [GeoPentech 2003-1] synthesized site-specific response spectra and seven pairs of spectrum-compatible acceleration histories were developed and peer-reviewed. The input histories had a typical duration of 30 to 40 seconds. To speedup analysis time without the loss of accuracy, the original records were shortened to have duration of ten to fifteen seconds for design process. The reduction in duration was done in such a way that all critical acceleration pulses were retained. The effect of conducting analyses with these shorter-duration acceleration histories is that the residual force and displacements in members might not be correctly estimated. The critical parameters for design process are the maximum responses in members. These maximum responses will be accurately captured with the shorter duration records. A full-length acceleration history was used at the end of the design process to verify the results. Table 1 lists the site-specific spectrum-compatible suite of acceleration records used in analysis; Figure 2 presents the site-specific spectrum and trace of a typical record.

Table 1. Acceleration records used in analysis

<table>
<thead>
<tr>
<th>Analysis</th>
<th>Record</th>
<th>Δt, sec</th>
<th>Time interval², sec</th>
<th>x-component ¹</th>
<th>y-component ¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>HIST01</td>
<td>Imperial Valley 1979</td>
<td>0.005</td>
<td>0-10</td>
<td>IV-FN</td>
<td>IV-FP</td>
</tr>
<tr>
<td>HIST02</td>
<td></td>
<td></td>
<td></td>
<td>IV-FP</td>
<td>IV-FN</td>
</tr>
<tr>
<td>HIST03</td>
<td>Sylmar 1994</td>
<td>0.02</td>
<td>0-12</td>
<td>FN-syl032</td>
<td>FP-syl032</td>
</tr>
<tr>
<td>HIST04</td>
<td></td>
<td></td>
<td></td>
<td>FP-syl032</td>
<td>FN-syl032</td>
</tr>
<tr>
<td>HIST05</td>
<td>Newhall 1994</td>
<td>0.02</td>
<td>0-12</td>
<td>FN-nwh032</td>
<td>FP-nwh032</td>
</tr>
<tr>
<td>HIST06</td>
<td></td>
<td></td>
<td></td>
<td>FP-nwh032</td>
<td>FN-nwh032</td>
</tr>
<tr>
<td>HIST07</td>
<td>Duzce 1999</td>
<td>0.005</td>
<td>2-14</td>
<td>FN-dzc265</td>
<td>FP-dzc265</td>
</tr>
<tr>
<td>HIST08</td>
<td>Kobe 1995</td>
<td>0.01</td>
<td>0-10</td>
<td>FN-taz320</td>
<td>FP-taz320</td>
</tr>
<tr>
<td>HIST09</td>
<td></td>
<td></td>
<td></td>
<td>FP-taz320</td>
<td>FN-taz320</td>
</tr>
<tr>
<td>HIST10</td>
<td>Superstitions Hill 1987</td>
<td>0.01</td>
<td>5-20</td>
<td>FN-b-pts217</td>
<td>FP-b-pts217</td>
</tr>
<tr>
<td>HIST11</td>
<td></td>
<td></td>
<td></td>
<td>FP-b-pts217</td>
<td>FN-b-pts217</td>
</tr>
<tr>
<td>HIST12</td>
<td>Imperial Valley 1979</td>
<td>0.005</td>
<td>2-12</td>
<td>FN-h-emo270</td>
<td>FP-h-emo000</td>
</tr>
<tr>
<td>HIST13</td>
<td></td>
<td></td>
<td></td>
<td>FP-h-emo000</td>
<td>FN-h-emo270</td>
</tr>
</tbody>
</table>

1. FN = Fault Normal and FP = Fault parallel
2. Time window from original record.
PERFORMANCE OBJECTIVE. The performance level for this building is the Life Safety (LS) performance level for a 475-year return seismic event. To reach this performance level, adequate supplemental damping (in the form of Fluid Viscous Dampers) was added to the existing nine-story superstructure to ensure that the existing steel and concrete moment-resisting frames would remain essentially elastic when subjected to site-specific input histories. The two main design objectives were to:

- Limit story drift ratios to less than 1.0 percent, and
- Limit the extent of flexural rotations in nonlinear elements as shown in Table 2.

For steel beams, member nonlinear flexural rotations were limited. This low limit was selected in order to prevent the type of non-ductile failure observed in the Northridge earthquake [SAC 1997]. Note that the limits of Table 2 are more conservative that those of FEMA 273[FEMA 1997] and FEMA 351 [FEMA 2000], because a more restrictive design guideline was selected by the design team [NYA&MI 2003] for this hospital building that is located in a region of high seismicity in California.

For the new damper frames, ASTM grade 50 A992 steel is specified. The as-built plans identified three types of concrete compressive strengths: 35 MPa (5 ksi) for columns, 28 MPa (4 ksi) for beams, and 21MPa (3 ksi) for walls. Grade 40 reinforcement was specified for all members, except for column main reinforcement which used Grade 60 steel. Independent material testing of selected steel members [Twining 2004] has shown that the as-built material properties for these elements did not deviate substantially from the values shown in the plans and specifications. AISC manuals [AISC 1980 and AISC 2001] were used to determine the nominal beam and column member sizes for modeling. A majority of steel beams and columns satisfied the flange and web

<table>
<thead>
<tr>
<th>Component</th>
<th>Allowable PH rotation, percent radian</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design guideline</td>
</tr>
<tr>
<td>Steel beam</td>
<td>0.5</td>
</tr>
<tr>
<td>Steel column</td>
<td>0.6</td>
</tr>
<tr>
<td>Concrete beam</td>
<td>0.5</td>
</tr>
<tr>
<td>Concrete column</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2. Acceptable performance limits for flexural hinge demands

ANALYSIS PROGRAM
To evaluate the seismic performance of the upgraded structure, a three-dimensional mathematical model of the structure was prepared using program SAP2000 [CSI 2004]. The model incorporated geometric (P-D), material (flexural yielding), and FVD (axial force-axial displacement hysteresis) nonlinearity. A benchmark model was developed to simulate the response of the existing building and assess the efficacy of FVD implementation. This model was used to conduct full nonlinear time history analysis, check the response of existing members, and determine the extent of nonlinear response in existing members. Parametric studies were also undertaken to investigate effects of directionality of torsional response, FEMA 273 (FEMA 1997) gravity preloads, effect of lower garage levels, and definitional properties of nonlinear flexural hinges.

All existing steel members were specified as ASTM A36 steel. For the new damper frames, ASTM grade 50 A992 steel is specified. The as-built plans identified three types of concrete compressive strengths: 35 MPa (5 ksi) for columns, 28 MPa (4 ksi) for beams, and 21MPa (3 ksi) for walls. Grade 40 reinforcement was specified for all members, except for column main reinforcement which used Grade 60 steel.
compactness requirements of AISC seismic provisions [AISC 2002]. All dimensions were specified as centerline-to-centerline. FEMA 351 [FEMA 2000] allows two approaches to model the panel zone: either the panel zone is explicitly model by specifying a rotational spring at the beam-to-column connection and beams are modeled to span to the face of the column, or panel zone flexibility is implicitly accounted for by modeling the beams to span to the centerline of the column. The latter approach was used in the analysis program described hereafter. At the roof level, some beam-to-column connections were modeled as pin connections, as indicated on the as-built plans.

For the baseline analysis, the columns extend one floor below the plaza level. In lieu of modeling the entire four-story garage structure below the superstructure, this simplified approach was used. Since the 250-mm perimeter shear walls provide lateral stiffness in these levels, the lateral stiffness in x- and y-directions were independently computed and modeled as linear spring. Similarly, the soil damping effect was modeled as linear dashpots at the lower levels. Soil-structure spring stiffness and dashpot damping properties were provided by GeoPentech [GeoPentech 2003-2]. Translational and rotational mass was placed at five percent eccentricity to the center of mass. Equivalent stiffness and damping for lower parking levels was modeled as links at the center of mass of level P1 and Plaza. The accuracy of this approach was verified by analysis, in which all garage floors were explicitly included; see Figure 3.

Figure 3. Mathematical model of the hospital building.

Program-default hinges were used, because plastic hinge rotations are limited to small amount for the upgraded structure. For verification, provisions of FEMA 273 [FEMA 1997] were used to separate a second model. The comparison of building responses validated that the program-default flexural hinges were adequate. Nonlinear flexural hinges were placed along the length of the member according to:

- At connection centerline for columns with weak panel zones (not expected to remain elastic)
- At the intersection of the beam flange to column flange for beams framing into column web
- At 1/3 depth of member from the centerline of support for all other members

FEMA 273 [FEMA 1997] gravity loads of 1.2D+0.25 L (unreduced) and 0.9 D were used. The inertial mass for each floor was modeled as a mass placed at five-percent eccentricity from center of mass in the x- and y-directions. The directionality of this eccentricity was investigated. Both lateral and rotational mass were included in the model. The total mass of the building, including the basement, was estimated at slightly over 26,000 Mg (57,000 kips).

ANALYSIS RESULTS

Global and local responses were used in evaluation. Analysis was conducted for 7 pairs aligned at 0 and 90 degrees. Response output for all 14 histories were extracted and averaged to obtain the desired seismic demand. Program Matlab [Mathwork 2001] was used to process the output data.

Response of the existing hospital building was evaluated using modal analysis, static nonlinear and linear and nonlinear dynamic procedures. The fundamental vibration periods of the existing building were approximately 2.8 and 2.5 sec in the x- and y-directions. Figure 4 presents the drift history response obtained from acceleration history analysis and the static pushover curve obtained from static nonlinear analysis. The maximum story drifts were close to 2%. At this level of story drift, roof displacement is approximately 24 in. and as such, the structure has lost most of its lateral capacity. Nonlinear hinge rotations in steel beams and columns exceeded 2% radian, and thus, both story drifts and member nonlinear flexural hinge rotations exceeded the design guideline limits. In addition, potential for soft story response at the first superstructure level (L3) existed. As such, the seismic response of the existing hospital building was unsatisfactory.

SEISMIC REHABILITATION PROGRAM

FVDs have been successfully utilized in upgrading the seismic performance of structures. This methodology is one of the recommended practices [FEMA 2000] and has been successfully implemented in both new construction [Miyamoto et. al, 2003-2] and in seismic rehabilitation [Miyamoto et. al. 2003-1] by the authors. The proposed seismic
rehabilitation scheme is to construct FVD frames on the exterior of the building. FVD frames would be detailed to provide fixity at the beam-column joints for these exterior frames. The FVDs—sized to control the drift—will be placed along the diagonals. The beams in the FVD frames will be attached to the existing perimeter beams by horizontal steel trusses that transfer the seismic forces from the existing diaphragm to the exterior frame. The basement concrete columns shall be increased in plan size for the first level to allow the exterior new FVD frame columns to bear on them. The FVD frame reaction will then be transferred to the existing column for the remaining levels of the parking structure. Figure 5 presents schematics of the building after rehabilitation.

Thirteen sizes of FVDs were used in analysis. To account for the manufacturing tolerances and variations in operating temperature the effective nominal damping coefficient (C) values were increased or decreased by 10-percent [Taylor 2004]. In one model, FVD damping coefficient was set to be ten percent below nominal. In a second simulation, FVD damping coefficient was set to be ten percent above nominal.

**RESPONSE OF THE RETROFITTED BUILDING**

The fundamental vibration periods of the retrofitted building were approximately 2.4 and 2.2 sec in the x- and y-directions. The retrofitted building is stiffer than the existing structure, since exterior moment frames were added to the building. Table 3 presents, the maximum roof displacement and base shear responses. These quantities were computed by obtaining the absolute maximum response of each fault-normal record and then averaging. The computed seismic base shear coefficient is approximately 5.5 percent of total weight. Typical roof displacement and base shear traces are depicted in Figure 6. Response of story drifts (Figures 4 and 6) show that the story drift is reduced by a factor of nearly two.

**Table 3. Average maximum response of the retrofitted building**

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Roof displacement, mm (in.)</th>
<th>Base shear, MN (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-direction</td>
<td>310 (12)</td>
<td>13.4 (3,000)</td>
</tr>
<tr>
<td>y-direction</td>
<td>270 (11)</td>
<td>14.3 (3,200)</td>
</tr>
</tbody>
</table>

Table 4 presents the story drifts in percent obtained from analysis. The entries in this table in each direction were computed by taking the average of the story drifts from the normal-fault components of motions. For example, the average drift in x-direction was computed by averaging values of records 1,2,3,5,7,11, and 13. Drift in the y-direction was computed by averaging drifts from records 2,4,6,7,10,12, and 14. For each record, the story drift response at level n was computed using
the exact formulation. As shown in Table 4, the computed drift demands are below or slightly above 1% radian and as such the design satisfies the requirements for drift limits. The entries of this table correspond to the computed drift at the center of mass of the structure.

The maximum seismic demand (force, velocity, and deformation) on FVDs was evaluated. Figure 7 presents the axial-force-axial displacement hysteresis response for a typical FVD obtained from analysis. Also shown in the figure is the hysteretic energy dissipated by this damper. This energy was obtained by integrating the area under the force-displacement curve. Note that significant seismic energy is dissipated by a single damper. Similar results were noted for all dampers. In the absence of dampers, the seismic energy absorbed by FVDs would have to be dissipated by the yielding of steel frame members, resulting in unacceptable large flexural rotations in these members.

Table 4. Computed story drifts (% radian).

<table>
<thead>
<tr>
<th>Direction</th>
<th>Roof</th>
<th>Mech.</th>
<th>L8</th>
<th>L7</th>
<th>L6</th>
<th>L5</th>
<th>L4</th>
<th>L3</th>
<th>L2</th>
<th>Plaza</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-</td>
<td>0.45</td>
<td>0.67</td>
<td>0.92</td>
<td>1.08</td>
<td>1.1</td>
<td>1.04</td>
<td>1.04</td>
<td>1.17</td>
<td>0.73</td>
<td>0.01</td>
</tr>
<tr>
<td>y-</td>
<td>0.36</td>
<td>0.59</td>
<td>0.84</td>
<td>0.94</td>
<td>1.01</td>
<td>0.97</td>
<td>0.92</td>
<td>1</td>
<td>0.75</td>
<td>0.02</td>
</tr>
</tbody>
</table>

Figure 7. Hysteretic response of a typical FVD unit
One of the primary objectives of analysis was to determine the pattern of plastic hinge formations and the level of plastic flexural rotations of the yielded members. Table 5 presents the maximum computed nonlinear flexural hinge rotation obtained from analysis and the limiting values set by the design guidelines. The retrofitted structure satisfies the design criteria requirements. Figure 7 presents the distribution of plastic rotations for the existing and retrofitted structures. For clarity, only the building elevation to level L5 is shown. Many hinges form in the existing building and a number of them exceed the life safety limits. In contrast, for the retrofitted building, fewer plastic hinges form and they were within the life safety limits.

For the upgraded structure, steel columns that might yield, are deformation-controlled (P_u / P_r <= 0.5), [FEMA 1997]. As such, formation of nonlinear hinges in these members is allowed.

Table 5. Maximum of average flexural rotations (% radian) in various components

<table>
<thead>
<tr>
<th>Component</th>
<th>Plastic hinge rotations, % rad</th>
<th>Allowable rotation, % rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel beam</td>
<td>0.42</td>
<td>0.5</td>
</tr>
<tr>
<td>Steel column</td>
<td>0.11</td>
<td>0.6</td>
</tr>
<tr>
<td>Concrete beam</td>
<td>0.27</td>
<td>0.5</td>
</tr>
<tr>
<td>Concrete column</td>
<td>0.19</td>
<td>0.5</td>
</tr>
</tbody>
</table>

**SUMMARY AND CONCLUSIONS**

Seismic response of a multi-story hospital building using unreinforced steel beam-to-column connections was evaluated by analysis. It was seen that the structure did not meet the Life Safety requirements of FEMA 1997. The structure was then modified by the addition of a limited number of FVDs. In the upgraded structure, the story drifts and member nonlinear rotational demands were below the target values specified in the design criteria. As such the upgraded structure meets the design criteria and is anticipated to perform well in seismic events.

**REFERENCES**


Miyamoto 2003-2, *US Practice – Design of Structures with Dampers*, Kit Miyamoto, Robert D. Hanson, Todd Kohagura, and Amir Gilani, proceedings of 13th World Conference on Earthquake Engineering, Vancouver, B.C.


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ABSTRACT

Codes and Standards are integral part of a structural engineer's tool-kit. In highway sector, for quite a few years now, perhaps caused by the transition from the working stress philosophy to limit state philosophy in IRC codes, heated debate on the complexity, content and size of codes has been going on. There are two distinct school of thoughts, one in favour of keeping the codes simple and prescriptive and the other in favour of bringing modernity in the codes and standards, at par with the international codes, which are growingly becoming more and more complex and voluminous. The important question that comes to our mind is whether the codes and standards in its present form is helping the structural engineers to develop their engineering skills or is it stifling their creativity? The objective of this paper is to share author's own experience, both as a code-user as well as a code-maker, on the status of present codes, what they should be, how they should be upgraded, how they should be treated in education, practice and how domestic research should affect them.

INTRODUCTION

Large scale construction and rapid urbanisation are an integral part of India's growth process. Unplanned, un-regulated and non-engineered structures are allowed to be built in India due to lack of regulation of engineering profession, which puts millions of lives, livelihoods and assets at great risk. Adoption of certain minimum standards and following the available codes of practices is therefore necessary, as it aims to mitigate this risk, and loss of investment in built environment. It helps manage multiple risks to infrastructure as well as increasing functionality in its day-to-day operation. The codes of practices for structural design influence the professional activities of all structural engineers. Whilst few will be involved with their creation, adaptation and updating, the great majority of practicing structural engineers are code-users, who will utilise their contents — some imaginatively and as an aid, some in a prescribed and unthinking fashion, a few in a defensive and negative way. Attitudes amongst users widely vary (supportive, puzzled, irritated, grateful etc). To the same individual they may well, at different times, be helpful, informative, restrictive or troublesome. Many user abuse the codal clauses, to suit their contractual position in a project, depending upon which side of the table they are in and which agency they represent.
Making the built environment resilient and robust requires not only constant development and upgradation of the codes and standards, but also ensuring that changes brought about in the codes is well understood by the practicing structural engineers, academia, as well as the software developers, so that the industry is able to quickly adapt themselves to the changing mode of development. This is of-course easier said than followed. It is a huge challenge and aiming to achieve this would require brain-storming sessions between all the stakeholders. Important issues that the government of the day, the policy makers, the academia, the structural engineering fraternity together must address in this respect are the following:

a. What is the role of Codes / Standards? Is there enough clarity on the subject to the practicing structural engineers?
b. Are the existing codes and standards easily understood by the practicing engineers and fully implemented? Is there any improvement needed in the way codes are developed?
c. Does our codes and standards stifle creativity? Should the format and content of codes be changed to promote lateral thinking?
d. Are our codes and standards at par with the international standards? If not, what needs to be done to narrow this gap?
e. Are our structures by and large code compliant? Is there any way by which we can ensure compliance to code and thereby fix accountability for design faults, if any?
f. Does India have the internal capacity (both in terms of quality and quantity) to quickly upgrade our codes and standards and keep pace with International standards? If not, what should be done to enhance capacity?

An attempt has been made by the author in this paper to address all the above issues on the basis of author's own experience.

ROLE OF CODES

Structural codes typically contain a mix of information, including factual data, design rules, advice on good practice and specific references elsewhere. Although their precise status varies in different parts of the world, it is generally accepted that the easiest way to demonstrate structural adequacy (and thus to gain an approval from client) is to show that the methods used in design agree with the provisions of the current local Code. Codes also comes very handy in case of any failure, where the court of law looks at the adequacy of designs through the prism of codal compliance route.

The legal status of Codes varies from country to country. It is enshrined in law in some countries, while it is merely advisory and optional in others. The perception about their legal standing is even more wide-ranging (and often misunderstood), with structural engineers often believing codes to be more influential than is actually the case. There is of course a considerable variation in the quality of codes in different countries, which varies from 'lucid', 'polished' and 'highly relevant and valuable' documents to 'unclear', 'ambiguous' and 'misleading' offerings.

Expectations from the code also widely varies depending upon the person who is looking at it. A practicing structural engineer with little insight into the subject will plead for simplicity in a code both for speed of application and to enable it to be used by engineers with limited experience. Some structural engineers would treat the code as text book and expect solution in the code for every problem that he is likely to face in design. Others with wide experience and having deep knowledge on the subject would expect the code to address only the fundamental knowledge when designing and would expect the code to give freedom for creativity and innovation in design. Those competing for worldwide markets would require the code to be at par with the international codes so that structural engineering practice becomes universal and practicing in global market becomes easy. Researchers and academicians on the contrary desire a code to be technically perfect and comprehensive, making use of the most recent research results. They would not like (over)simplification in the code.

The correct and ideal approach for any structural engineer, should be to treat the available codes and standards as only 'one of the many' portfolio of material, which is to be used for structural design. Designers are expected to understand the background of codal clauses, to look beyond the code as they grow in the profession and also read international codes of practices to better understand the codal provisions of the country. Manufacturers' literature, design guides, computer software, textbooks, manuals, volumes of worked examples etcall should form a part of structural engineers tool-kit while performing designing and detailing.

DEVELOPMENT PROCESS FOR CODES & STANDARDS IN INDIA

In India, there are several code making bodies such as:

a) Bureau of Indian Standards (BIS),
b) Indian Roads Congress (IRC) and
c) Indian Railways (IR).

Bureau of Indian Standards (BIS), the national standards body of India, which has published more than 18000 Indian Standards, and cover the total scope of code making activities. Indian Railways also develop codes concerning their own constructions and use. Indian Roads Congress (IRC) is responsible for all codes dealing with with surface transport by road including the structural codes. In India, the design codes are written mostly by comparing and updating their knowledge contents borrowing from foreign codes – viz. British, American, Australian, etc. Basic theoretical work or practical research for confirming the applicability of underlying assumptions (e.g. Load and material factors) for Indian conditions is rarely done in a proactive way by researchers, academicians, forcing the code committees to base their codes relying on the published data.

In contrast Europe has taken great strides in unification and harmonisation of their codes with the objective of free trade within the European union. Euro code is now being followed by 26 European countries. Unified codes have helped engineers to work all over Europe without having to familiarize with a different code in each country.

Author is of the firm view that there should be a single agency developing the codes and standards in India. Multiple agencies working on the same topic results in unnecessary conflicts, duplication of efforts, which is undesirable. A classic example is seismic code in India. At present there are 4 set of seismic codes (i.e. developed by BIS, IRC, IRS and RDSO). All these set of codes gives different set of results for the same problem. A structure which is designed as per IRC code may not be safe, when checked by BIS code or by IRS code or by RDSO code. This is avoidable, if all Codes are unified.

Code development requires best talent from practitioners, academicians, manufacturers, contractors, and other stakeholders. The code is developed through consensus. Being a voluntary work, it becomes difficult to attract talented people to invest lot of time in code making. Even when talented people join the committee, they do not devote quality time & attention that is required to be devoted for code development. This is reflected in the quality of the document codes we develop. The author is of the view that experts involved in code development process must be made accountable and also compensated financially for their time and involvement.

ISSUES WITH CODE COMPLIANCE

Achieving a structure which is code compliant and which serves its full design service life is not an easy task. It is like a 3legged Race. Developing good codes and standards alone will not be sufficient to achieve the ultimate objective of getting a safe structure. Effective implementation of the codes and standards in practice needs to be ensured.

Codes are becoming more and more voluminous and complicated which is a deterrent for many structural engineers to follow the codes. There is a need therefore to make the codes more user friendly. Further, codes should be supplemented by Designer's Guides, Explanatory Handbooks, User Manuals, Worked Examples ...etc. to help ensure compliance. Also workshops, seminars and conferences need to be organised Professional associations (like IAStructE, ICI, ACCE, CEAI, INSDAG ...etc.) or reputed consultants may be engaged by the Code making bodies in preparing such manuals for the users.

CONCLUSIONS

Structural codes have been and may be expected to continue to be an important feature in the work of professional structural engineers. They require qualitative upgradation on a regular basis. Whilst it might be argued that the profession gets the Codes (and code support) it deserves, the influential nature of such documents means that there is a collective responsibility on the profession to strive for an ever improving planning, process and product, leading to better practicality, performance and profitability.

REFERENCES

1. Presentation of Alok Bhowmick titled "Implementation of Codes and Standards in India's Transport Sector"at the Round table on "Standards for Infrastructure Development in India's Transport Sector" organised by National Institute of Public Finance & Policy

2. Paper titled "DO STRUCTURAL CODES STIFLE CREATIVITY?" by D. A. Nethercot, Department of Civil and Environmental Engineering, Imperial College London, published in IABSE Spring Conference, Kuala Lumpur, April 2018
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Waste-derived Manufactured Aggregate For Concrete Permitted In IS: 383 – 2016

Dr. A.K. Mullick
Former Director General,
National Council for Cement and Building Materials

ABSTRACT

Taking note of non-availability and difficulty in having coarse and fine aggregate for concrete from natural sources in some parts of the country and need of effective management of waste materials, BIS has recently permitted use of recycled concrete aggregate (RCA), iron and steel slag, copper slag and bottom ash. These have been called 'manufactured' aggregate in IS:383-2016. The decision is in line with results of copious R&D carried out in the country, some of which are described in the paper.

Keywords: Aggregate, concrete, C&D wastes, industrial wastes, standardisation.

INTRODUCTION

Aggregates, coarse and fine, occupy nearly 75 percent of volume of concrete. Next to water, concrete is the most widely used material the world over. One of the reasons for this wide acceptance is that the ingredients of concrete are either naturally occurring, or made from naturally occurring raw materials. Industrial waste materials also find increasing acceptance in making cement and concrete.

Most project specifications prescribe aggregate from natural sources. In particular, it is required that coarse aggregate shall consist of clean, hard, strong, dense, non-porous and durable pieces of crushed stone, crushed gravel, natural gravel or combination. Fine aggregate shall be natural sand, crushed stone, crushed gravel and combination, and Grading of Zone I, II and III are preferred.
Great challenges are being faced in obtaining quality aggregates from natural sources for concrete constructions [1]. In particular, there are restrictions on the use of river sand because of environmental and ecological concerns of erosion of soil in the river bed, change in the course of the river and stability of the banks, causing floods. This led BIS to permit alternatives in the recent revisions of IS: 383 in 2016. A summary of recommendations is extracted in Table 1. The decision was based on proposals emanating from copious indigenous research [2]. The relevant R&D is discussed in this paper.

**RECYCLED CONCRETE AGGREGATE**

Recycled concrete aggregate is obtained by processing construction and demolition (C&D) wastes [3]. Recent revision of IS: 383 recognises use of C&D wastes for obtaining aggregates as a step towards effective management and utilisation of this waste. This however, requires necessary processing and care to ensure their efficacy in their use in concrete. These aggregates may be of two types; Recycled Concrete Aggregate (RCA) derived from concrete rubbles after proper processing, and Recycled Aggregate (RA) made from C&D waste comprising concrete, brick, tiles etc., [3].

Recycled concrete aggregate (RCA) contain not only the original aggregate, but also hydrated cement paste. This paste reduces the specific gravity and increases the porosity compared to similar virgin aggregates. Higher porosity of RCA leads to a higher water absorption. Recycled aggregate will typically have higher absorption and lower specific gravity than natural aggregate [3].

The main influence of RCA is on the strength characteristics of concrete made with it, which is generally lower than that those made with virgin aggregate. The reason for the loss of strength is usually associated with [4]:

- The weaker interfacial transition zone (ITZ) between the aggregate phase and the mortar, due to the aggregate already having a coat of weak mortar attached on its surface, and
- This attached mortar raising the porosity of the resultant concrete.

Table 1. Extent of Utilisation of Manufactured Aggregate (Table 1, IS:383-2016)

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Type</th>
<th>Extent of Utilisation, %</th>
<th>Plain Cement Concrete</th>
<th>RCC</th>
<th>Lean Concrete (= M15 Gr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Coarse Aggregate</td>
<td>Iron slag</td>
<td>50</td>
<td>25</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Steel slag</td>
<td>25</td>
<td>Nil</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RCA</td>
<td>25</td>
<td>20 (= M25 Gr)</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RA</td>
<td>Nil</td>
<td>Nil</td>
<td>100</td>
</tr>
<tr>
<td>2.</td>
<td>Fine Aggregate</td>
<td>Iron slag</td>
<td>50</td>
<td>25</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Steel slag</td>
<td>25</td>
<td>Nil</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Copper slag</td>
<td>40</td>
<td>35</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RCA</td>
<td>25</td>
<td>20 (= M25 Gr)</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bottom Ash</td>
<td>nil</td>
<td>nil</td>
<td>25</td>
</tr>
</tbody>
</table>

In fact, all the properties of concrete dependant on the surface characteristics of coarse aggregate are affected. Such properties are; compressive and tensile strength, workability, elastic modulus; and durability characteristics like water absorption, sorptivity, drying shrinkage, abrasion resistance and chloride ion penetration [5].

**R&D on use as Coarse Aggregate**

All the above-mentioned properties of concrete containing recycled concrete aggregate can be improved by proper processing of the RCA. Recent research on use of recycled concrete aggregate in M75 grade concrete, after due processing and improved mixing techniques have been published (4, 5). The effects of removal of adhered mortar are summarised below.

Materials used in this investigation were Portland pozzolana cement conforming to IS: 1489 – Part I, fine aggregate (river sand) and virgin coarse aggregate (10mm and 20mm sizes) conforming to IS: 383, Silica fume conforming to IS: 15388 and...
Superplasticiser (PCE based) conforming to IS: 9103. Water used was from municipal supply (IS: 456). The control concrete mix was of M75 grade having target 28-day strength of 82 MPa and 120 ± 10 mm slump. The mix proportions are in Table 2.

Table 2: Mix Proportions

<table>
<thead>
<tr>
<th>Materials</th>
<th>Quantity, kg/cum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (PPC)</td>
<td>500</td>
</tr>
<tr>
<td>Silica fume</td>
<td>35</td>
</tr>
<tr>
<td>Water</td>
<td>130</td>
</tr>
<tr>
<td>Fine aggregate</td>
<td>680</td>
</tr>
<tr>
<td>Coarse aggregate</td>
<td>1150</td>
</tr>
<tr>
<td>Superplasticiser</td>
<td>8 (1.6% bwoc)</td>
</tr>
<tr>
<td>w/b ratio</td>
<td>0.243</td>
</tr>
</tbody>
</table>

Varying amounts of coarse aggregate from natural sources were replaced by RCA in steps of 10 percent, up to 100 percent. Demolished concrete boulders (size ranging from 100 to 300 mm) from 10 years old 300 mm thick rigid pavement were collected. Selected boulders were made free from any contaminants. A crushing plant for producing crushed-rock aggregate, comprising primary jaw crushers and secondary cone crushers and screens, was used to produce coarse RCA in two size fractions, 20-10 mm and 10-5 mm (Figure 1).

The recycled aggregate (RCA) from the crushing plant was further processed in a Los Angeles abrasion test machine by using steel balls and rotated a total number of 700 revolutions @ 33 rpm for each batch of total weight 10 kg (5 kg each for two sizes of 20-10 mm and 10-5 mm). Such processed material was discharged from the machine and washed manually with water properly, till the water after washing was clean. The washed material was air dried and stacked in the laboratory before use in the experimental programme (Figure 2). All the aggregates including processed RAC were used in saturated surface-dry condition.

Fig. 1 Unprocessed recycled aggregate

Tests on hardened concrete were carried out for ages up to 180 days; results up to 28 days are discussed here. Till the age of testing, the specimens were moist-cured. Typical results of compressive, flexural and split tensile strength of concrete with different proportions of recycled concrete aggregate, which were subjected to 500 revolutions, are shown in Table 3 (5).

In so far as the effect of processing was concerned, it was observed that all the properties of concrete improved with the number of revolutions in LA machine, for any level of replacement by RCA. The workability and strength of concrete decreased as the proportion of RCA increased. Compared to reference concrete with virgin aggregate, 140 mm slump gradually decreased to 80 mm with 100 percent RCA; similarly, the 28-day compressive strength gradually decreased from 85.4 MPa to 62.3 MPa. In effect, it will mean that 60 MPa concrete with 80 mm slump was possible with 100 percent processed RCA. However, the decrease in durability parameters (reported in Ref. 5) was more pronounced, which will restrict the proportion of replacement of virgin aggregate by processed RCA.

Use of Fine Fractions as Fine Aggregate

The fine fractions obtained during processing of concrete rubbles can be used as fine aggregate. Use of finer fractions (< 4.75 mm) of recycled concrete as part replacement of fine aggregate from natural sources was investigated. The results...
have been published recently [6]; as such, only salient findings are reported here.

The < 4.75mm fraction obtained was brought into grading zone II confirming to IS 383-1970, by suitably mixing. Properties of fine aggregate from natural sources and recycled concrete are compared in Table 4. The quality of recycled aggregate is judged in terms of materials content, density and water absorption [3]. In many specifications, the limits of specific gravity (minimum) and water absorption (maximum) are 2.2 and 5 percent (South Korea) or 2.2 and 7 percent (Japan) respectively. If similar specifications are adopted in India, the present sample of recycled fine aggregate will satisfy the same.

Table 3. Concrete Strength Properties with processed RCA (4)

<table>
<thead>
<tr>
<th>RCA, (500 rev.)</th>
<th>Compressive strength (MPa)</th>
<th>Flexural strength (MPa)</th>
<th>Split tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7- day</td>
<td>28- day</td>
<td>56- day</td>
</tr>
<tr>
<td>0%</td>
<td>70.69</td>
<td>85.43</td>
<td>93.81</td>
</tr>
<tr>
<td>10%</td>
<td>65.71</td>
<td>81.92</td>
<td>89.69</td>
</tr>
<tr>
<td>20%</td>
<td>63.04</td>
<td>80.51</td>
<td>87.90</td>
</tr>
<tr>
<td>30%</td>
<td>59.41</td>
<td>78.34</td>
<td>85.68</td>
</tr>
<tr>
<td>40%</td>
<td>58.43</td>
<td>75.40</td>
<td>83.22</td>
</tr>
<tr>
<td>50%</td>
<td>55.52</td>
<td>73.23</td>
<td>82.01</td>
</tr>
<tr>
<td>60%</td>
<td>54.76</td>
<td>71.30</td>
<td>80.60</td>
</tr>
<tr>
<td>70%</td>
<td>52.23</td>
<td>68.79</td>
<td>79.30</td>
</tr>
<tr>
<td>80%</td>
<td>50.00</td>
<td>65.43</td>
<td>78.37</td>
</tr>
<tr>
<td>90%</td>
<td>47.96</td>
<td>63.03</td>
<td>75.41</td>
</tr>
<tr>
<td>100%</td>
<td>45.89</td>
<td>62.30</td>
<td>74.82</td>
</tr>
</tbody>
</table>

Table 4. Properties of fine aggregate from RCA (6)

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Material</th>
<th>Water Absorption</th>
<th>Specific Gravity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Natural Sand</td>
<td>0.21</td>
<td>2.6</td>
</tr>
<tr>
<td>2</td>
<td>RCA</td>
<td>6.2</td>
<td>2.41</td>
</tr>
</tbody>
</table>

Two control concrete mixes viz. M30, M40 grades were designed using virgin aggregates. Further mixes were obtained by replacing the fine aggregate fraction with RCA in steps of 10 percent, up to 50 percent. No other changes were made.

The target slump of 75 mm was reduced to 65 mm (M30 grade) and 59 mm in M40 grade, when 50 percent of natural sand was replaced by RCA. This lowering of workability can be offset by use of chemical admixtures and using the aggregate in saturated surface dry condition. Compressive strength of concrete of M30 and M40 grades were continuously decreasing with the increase in replacement percentages of natural sand by RCA. Results for M40 grade concrete are shown in Figure 3.

![Figure 3. Compressive strength of M40 concrete with sand replaced by RCA (6)](https://example.com/image.png)
The compressive strength was lower by 11.3 percent for M30 grade and 7.1 percent in case of M40 grade at 28 days, when 20 percent of natural sand was replaced by recycled aggregate [6].

**IRON AND STEEL SLAG**

IS: 383-2016 allows use of iron and steel slag as alternate coarse and fine aggregate. Iron slag (Blast Furnace slag) is obtained as by-product during manufacture of iron in blast furnace or basic oxygen furnace in integrated iron and steel plant. It can be air-cooled or granulated; the latter being lightweight, should be processed further to have bulk density > 1.3 kg/l for use in normal weight concrete. These must satisfy test for 'iron unsoundness' specified in the specification.

Steel slag is a by-product obtained in the manufacture of steel in integrated iron and steel plants. Metallic iron contained in the slag is removed by magnetic means. It is obtained in air-cooled form and must be weathered to free lime content within limits. In addition, slag aggregates must satisfy the limit of 'volume expansion ratio' when tested as per the procedure prescribed in IS:383-2016.

**Use as Coarse Aggregate**

Tests were conducted on use of BF slag as part replacement of natural coarse aggregate in high strength concrete (7). The mix used 447 kg/m$^3$ cement, w/c ratio 0.37 and 7 % silica fume. Superplasticiser was used @ 0.8 – 1.0 %, to render satisfactory workability. The compressive strength and split tensile strength were of the same order as the reference mix, up to 30 % replacement by BF slag (Table 5).

Table 5. Trials with BF slag as coarse aggregate (7)

<table>
<thead>
<tr>
<th>Mix</th>
<th>Natural Coarse Aggregate replaced by BF slag, %</th>
<th>28 – day compressive strength, MPa</th>
<th>28 – day split tensile strength, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0 (Reference mix)</td>
<td>69.16</td>
<td>3.17</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>69.54</td>
<td>3.50</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>70.03</td>
<td>2.17</td>
</tr>
<tr>
<td>4</td>
<td>30</td>
<td>71.52</td>
<td>3.30</td>
</tr>
</tbody>
</table>

**Use as Fine Aggregate**

Coarser particles of BF slag of sand sizes have been envisaged as part replacement of natural sand in concrete (2). Starting with larger pieces of granulated slag as feed material, it is crushed and sieved in a manner similar to obtain crushed rock sand.

The resulting product conformed to grading zone II of IS: 383 (Figure 4) and particle shapes were similar to river sand (subangular to sub-rounded). Any excess of elongated particles will need processing in a Vertical Shaft Impactor (VSI) crusher.

Unlike crushed rock sand, the proportion of fine fractions (< 150µ) was comparable to natural sand (<4.6%).

Results of compressive strength of M40 grade concrete, in which natural sand was replaced in full or part by slag sand, as well as combinations with crushed rock sand or crusher dust are given in Table 6 (2). Addition of slag sand improved the compressive strength. Workability was satisfactory; in fact, mixing with crushed sand helped in achieving favourable proportion of fine fractions.

**COPER SLAG AS FINE AGGREGATE**

Sand-sized fraction of copper slag has been investigated as part replacement of natural sand in concrete [8]. In this investigation, copper slag conformed to grading limits, but the size fraction below 1 mm was greater than in natural sand (Figure 5).

The concrete mixes had cement content 384 Kg/m$^3$, water/cement ratio 0.6, coarse aggregate and fine aggregate contents 1136 and 712.6 kg/m$^3$ respectively constant, except that the fine aggregate was made up of natural sand being replaced by copper slag in increments of 20 percent. Results of slump and compressive strength are shown in Table 7 (8).

![Figure 4. Comparison of Sieve analyses of natural sand (NS) and slag sand (GBFS) (2)](image-url)
Table 6. Compressive strength data on M40 grade concrete with slag sand (2).

<table>
<thead>
<tr>
<th>Fine Aggregate</th>
<th>7 day Strength, MPa</th>
<th>28 day Strength, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>100% Natural Sand</td>
<td>38-42</td>
<td>48-52</td>
</tr>
<tr>
<td>100% crushed rock sand</td>
<td>40-44</td>
<td>50-54</td>
</tr>
<tr>
<td>100% GBS</td>
<td>42.9</td>
<td>53.3</td>
</tr>
<tr>
<td>50% GBS + 50% NS</td>
<td>39</td>
<td>52</td>
</tr>
<tr>
<td>50% GBS + 50% crushed rock Sand</td>
<td>40.8</td>
<td>52.3</td>
</tr>
<tr>
<td>50% GBS + 50% Crusher Dust</td>
<td>30.9</td>
<td>49.7</td>
</tr>
</tbody>
</table>

Table 7. Tests on concrete mixes with copper slag

<table>
<thead>
<tr>
<th>Mix</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>% of copper slag in fine aggregate</td>
<td>0</td>
<td>20</td>
<td>40</td>
<td>60</td>
<td>80</td>
<td>100</td>
</tr>
<tr>
<td>Slump, mm</td>
<td>43</td>
<td>44</td>
<td>58</td>
<td>85</td>
<td>Collapse</td>
<td>Collapse</td>
</tr>
<tr>
<td>Compressive strength, MPa</td>
<td>36.6</td>
<td>35</td>
<td>33.4</td>
<td>32</td>
<td>30.6</td>
<td>28.4</td>
</tr>
<tr>
<td>Flexural strength, MPa</td>
<td>3.89</td>
<td>4.04</td>
<td>4.45</td>
<td>4.58</td>
<td>4.43</td>
<td>4.48</td>
</tr>
</tbody>
</table>

It can be seen that replacement of natural sand by copper slag increased the slump. This was ascribed to the glassy surface of copper slag, which absorbed less water (0.17%) than natural sand. This led to excess of free water forming internal voids, contributing to increased bleeding and porosity, weakening the strength. Gradual decrease in compressive strength with increase in copper slag percentage was noted. Flexural strength was not much affected.

**BOTTOM ASH AS FINE AGGREGATE**

Bottom ash and fly ash are the by-products of the combustion of pulverised coal from thermal power plants. Bottom ash is the coarser material, which falls into the bottom of the furnace and constitute about 20 percent of total ash content of the coal fed into the boilers. In view of substantial amount generated each year and already accumulated mixed with pond ash, it merits consideration.

Results of a comprehensive experimental investigations on use of bottom ash in cement and concrete, both as pozzolanic additive, and as replacement of natural sand as fine aggregate, has been reported [9]. In this series of tests, as-received bottom ash was used in part replacement of natural sand as fine aggregate. It conformed to grading zone II of IS: 383, had lower density and higher water absorption. At replacement levels up to 20 percent, the combination with natural sand was within the stipulated grading limits of zone II (Figure 6).

The details of concrete mixes and results of slump and compressive strength are in Table 8. In order to maintain the same workability at the same water content, the dosage of the superplasticiser had to be increased from 1% (in case of river sand) to 1.75% in case of 20 percent bottom ash. It was noticed that with 20 percent bottom ash replacing sand, the compressive strength of concrete at 28 days and later were greater than with natural sand alone [9]. This was possibly due to pozzolanic property of bottom ash, albeit much lower than fly ash from the same source.
CONCLUSIONS

Recently revised IS: 383-2016 has permitted 'manufactured' sand obtained from C&D wastes and other industrial wastes as part replacement of coarse and fine aggregate from natural sources. These can be used in plain concrete, lean concrete (≤ M15 Grade) and reinforced concrete (≤ M25 Grade). The research back-up is described in the paper. Manufactured aggregate should fully conform to the quality requirements and the extent of utilisation stipulated in IS: 383-2016.

Table 8. Mix proportions with bottom ash and test results

<table>
<thead>
<tr>
<th>Mix</th>
<th>Cement, kg/m³</th>
<th>w/c</th>
<th>Sand, %</th>
<th>Slump, mm</th>
<th>Comp. Strength, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7d</td>
</tr>
<tr>
<td>IA</td>
<td>500</td>
<td>0.37</td>
<td>100</td>
<td>894</td>
<td>6.96</td>
</tr>
<tr>
<td>IB</td>
<td>80</td>
<td>73</td>
<td></td>
<td></td>
<td>45.3</td>
</tr>
<tr>
<td>IIA</td>
<td>450</td>
<td>0.4</td>
<td>100</td>
<td>944</td>
<td>0.85</td>
</tr>
<tr>
<td>IIB</td>
<td>80</td>
<td>81</td>
<td></td>
<td>2.1</td>
<td>0.26</td>
</tr>
<tr>
<td>IIIA</td>
<td>400</td>
<td>0.43</td>
<td>100</td>
<td>863</td>
<td>0.55</td>
</tr>
<tr>
<td>IIIB</td>
<td>80</td>
<td>78</td>
<td></td>
<td>55.0</td>
<td>29.4</td>
</tr>
</tbody>
</table>

REFERENCES


**Shiva Statue at Nathdwara Rajasthan**

**Engineer Dr. Abhay Gupta**

---

**FACT FILE**

**CLIENTS NAME:** TATPADAM UPVAN, MIRAJ GROUP NATHDWARA( RAJ)

**SCULPTOR:** Mr. NARESH KUMAWAT, TEMPLESMAC, GURGAON

**STRUCTURAL CONSULTANT:** SKELETON CONSULTANTS PVT. LTD.

**PROJECT COST:** INR 100 Crores

**CONSTRUCTION STATUS:** ON GOING

**PROJECT BRIEF:**

A Gigantic 351-feet high Shiv statue is under construction at the top of a 50m high hillock in the sitting position at Nathdwara near Udaipur in Rajasthan. The base of the main statue approx. 60mx45m rectangular shape is representing a hill top.

This is one of the world's tallest in-situ concrete statues, being fourth after Statue of Unity India 182m, Spring Temple Buddha China 153m, Ushiku Daibutsu Japan 120m, this is 107m tall. As per mandate from project Sponsors MIRAJ group Nathdwara, the design life is 250+ years.

**STRUCTURAL GEOMETRICS**

The shape of statue is irregular & unsymmetrical. It falls in high importance factor category as per IS1893.

Wind tunnel testing has been done in simulated wind environment on scaled model of statue and surrounding topography, at WindTech Consultants, Sydney Australia. The
analysis of skin under the self-weight and wind forces is done considering the thermal stresses relevant to temperature changes at Nathdwara.

Statue is designed as hybrid Structure. Structural system consists of framework of Steel columns and beams surrounding concrete cores of shear walls in such a manner that the stiff vertical core right up to top is available. High strength (350Mpa) parallel flange rolled steel sections like UB, UCs are used. All connections have been made using High Strength Friction Grip bolts with galvanization. About 2500MT of structural steel sections (parallel flange) are being used to erect the frame which is varying vertically and horizontally in the shape of statue

There will be a 170 feet (51m) TRISHUL free standing by the side of the statue above 120 feet high sitting platform. There will be a concrete shell skin outside with hollow space inside which will house a lift and stairs for maintenance purpose along with few floors for assemblies.

Project is in the construction stage and likely to be complete by end of 2018. Total weight of statue shall be approx. 25000MT. Er. Ms. Vandana Verma is the lead Designer at Skeleton under the guidance of Dr. Abhay Gupta and Prof. Prem Krishna of IIT Roorkee is peer review consultant. Shilpkar is Mr. Naresh Kumawat, Templesmac, Gurgaon & Contractors are M/s Shapoorji Pallonji co. Pvt. Ltd., Mumbai.

Contact
Skeleton Consultants Pvt. Ltd.
A-75, II Floor, Sector-5, Noida-201301 (U.P)
Tel.: 0120-4222642
ABSTRACT:
When liquefaction occurs, the soil loses its stiffness and strength and can not only deform but also flow laterally. In-situ testing is relied upon to assess the liquefaction potential of soils due to the difficulties in obtaining and laboratory testing of undisturbed samples. The Standard Penetration Test (SPT) and the Cone Penetration Test (CPT) are the two most frequently used field investigations for determining the characteristics of soils. In this paper SPT values have been used as they remain by far the most popular and economical method of subsurface investigation in India. The methods suitable for design offices in the Indian context have received special attention in this paper which examines the effects of liquefaction on piled bridge foundations.

Keywords: Liquefaction, Inertial Effects, Kinematic Effects, Evaluation

1. Introduction
IS 1893 Part 1 (2016) defines liquefaction as a state primarily in saturated cohesionless soils wherein the effective shear strength is reduced to negligible value for all engineering purposes, when the pore pressure approaches the total confining pressure during earthquake shaking. In this condition the soil tends to behave like a fluid mass.

IS 1893 Part 1 (2016) stipulates the types of strata that should be investigated to evaluate its potential for liquefaction.

The Caltrans Geotechnical Manual (2014) suggests that soils with the characteristics shown in Table 1 are not liquefiable.
(N)_{60} is defined in a subsequent section of this paper.

Table 1

<table>
<thead>
<tr>
<th>(N)_{60}</th>
<th>% Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;30</td>
<td>≥5%</td>
</tr>
<tr>
<td>&gt;25</td>
<td>≥15%</td>
</tr>
<tr>
<td>&gt;21</td>
<td>≥35%</td>
</tr>
</tbody>
</table>

Empirical and semi-empirical approaches for determining potential of liquefaction used in design offices cannot account for all the effects attributable to the characteristics of the soil, the topography, the ground motion and the structural arrangement. Nevertheless, for design office applications such approaches are invaluable especially if there is a consensus in a large body of experts and investigators that a particular approach is acceptable with the present state of knowledge.

A review of the design codes applicable to piles in liquefiable soils is available at Ghosh et al (2012).

A comprehensive compilation of the SPT based procedures including case history databases are recorded by Idriss and Boulanger (2010).

Wells (Caissons) and Piles are the most common types of foundations used in bridge structures. They have the advantage of transmitting loads to competent strata below when the upper strata has been subjected to liquefaction during strong ground shaking. Liquefaction results in loss of surface friction and stabilizing support of the surrounding soil over the depth affected by the phenomenon. Well foundations, due to their large size and stiffness are able to better cope with the effects of liquefaction.

2. Liquefaction Basics

The shear strength of cohesionless soil, \( \tau \), depends mainly on the angle of internal friction and the effective stress acting on the soil grains and can be expressed as

\[
\tau = \sigma' \tan \phi \tag{1}
\]

\[
\sigma' = \sigma - u \tag{2}
\]

where \( \tau \) = shear strength, \( \sigma' \) = effective normal stress, \( \sigma \) = total normal stress, \( u \) = pore pressure, \( \phi \) = angle of internal friction.

When saturated loose cohesionless soils are subjected to earthquake loading, they tend to settle due to the densification of soil. The duration of the cyclic stress application is so short compared to the time required for water to drain, that excess pore pressure progressively builds up. When the pore pressure equals the total stress, thereby reducing the effective stress to zero, the soil will experience a sudden degradation of strength and stiffness.

The most popular and accepted method of analysis of soil substrata for liquefaction potential is available at Youd et al (2001). This method is suited upto a depth of 23m below ground. Extrapolation beyond this depth could be of uncertain validity, AASHTO (2012).

The earthquake magnitude to be considered for liquefaction potential analysis is slightly different in international codes. AASHTO (2012) stipulates the basis as a 975 year return period (i.e., 5% possibility of exceedance in 50 years). IS 1893 Part 1 (2016) is not based on Probabilistic Seismic Hazard Analysis at present but stipulates specific values of horizontal ground accelerations to be adopted in the four seismic zones of the country.

3. Piled Bridge Foundations

The diameter of piles in bridge foundations are usually restricted to the range 800 mm to 1500mm for reasons of economy and construction convenience, though occasionally large diameter (2000mm to 2500mm) or raked piles have been adopted in India. As a matter of interest, for the 6.15 km long rail-cum-road Padma Bridge in Bangladesh some foundations consist of racked (inclination 1+1: 6V) steel tubular driven piles, 6 nos, each of 3m diameter and 128m depth. These were necessitated by the susceptibility to deep sour and high seismicity of the area. Since the surrounding soil over the liquefied height can no longer be depended upon to provide lateral support to the pile, the resulting deformations (including P-delta effects) and bending effects can be quite significant. Example of a bridge foundation on piles is shown in Fig. 1. This part of the analytical process relates to the "inertial effects" of liquefaction.

![Fig. 1 3D Model of Bridge Pier and Piled Foundation](image-url)
The diameter of pile must be selected with care so that apart from its vertical load carrying capacity the serviceability of the structure is not affected. The potential consequences of liquefaction associated with pile foundations include loss of vertical load capacity, loss of lateral stiffness and capacity, lateral loading due to lateral soil displacements, and down drag on piles due to post-liquefaction reconsolidation of soil.

Fig. 2 from Bhattacharya et al (2005) depicts deflected shapes of pile and pier in some structural configurations.

**Fig. 2 Effective Lengths for Buckling Considerations**

The problems concerning liquefaction have another dimension. Lateral spreading arises if soils subject to liquefaction are situated on a slope or near a river channel or sea which may cause movement of the liquefied soil perpendicular to the waterfront thereby aggravating the induced effects further. In these conditions the presence of non-liquefied soil strata overlying the liquefiable soil makes the situation even more onerous. The non-liquefied crust would exert passive earth pressure on the foundation. For shallow slopes a simplified prescriptive approach is indicated in JRA (1996, 2002) based on back calculations from the observed damages in the Kobe earthquake of 1995. The equivalent static forces acting on the bridge foundations due to the ground flow can be estimated as (a) passive earth pressure of the upper non-liquefiable layer, plus (b) 30% of the overburden pressure of the lower liquefiable layer, as shown in Fig. 3. This part of the analytical process relates to the "kinematic effects" of liquefaction.

Fig. 3 JRA (1996) code of practice showing the idealization for seismic design of bridge foundation

Showa Bridge, Fig. 4, is an example of failure due to lateral spreading resulting in drag down the slope in the 1964 Niigata Earthquake. The bridge has a 24.8m wide deck and a total length of 303.9 m (13.75 m + 10 @ 27.64 m + 13.75 m). Each of the pier foundations consists of 9 piles in a single row. One of the investigators, Bhattacharya et al (2005), identified buckling as a possible failure mechanism of the piled foundations.

Following the 1964 collapse, a law was passed to prevent bridge piers being founded on a single row of piles, Bhattacharya et al (2005).
The centrifuge test results of Haigh and Madabhushi (2002) suggest that the pressure distribution in Fig. 3 is under-conservative in the transient phase but gives reasonable predications for residual sliding.

The existing simplified methods cannot account for the "inertial effects" and the "kinematic effects" of liquefaction as coupled phenomena. This is justified by the fact that peak inertial loads are likely to occur before ground flow Kavazanjian et al. (2011). The response of the structure to liquefaction is checked separately for the peak inertial load and the kinematic load (where applicable) without superimposing or adding the two.

Piles must be checked for buckling instability due to both Inertial and Kinematic effects as well as out-of-straightness which would increase lateral deflections thereby reducing the buckling load. As per JRA (1996, 2002) the effective length of the pile should in no case exceed 50.

In the case of stiff piers including plate piers it becomes difficult to avoid plastic hinging in the piles. Pile integrity and ductile behaviour should be ensured in such cases. The potential hinge locations should be provided with proper ductile reinforcement, for instance, in accordance with IS 13920 (2016) at the following locations: a) At the pile heads just below the pile cap, b) at the depth where maximum bending moments develop in the pile (c) at the interfaces of soil layers having markedly different shear deformability. For the locations (b) and (c), longitudinal as well as confining reinforcement of the same amount as that required at the pile head should be provided.


4. Quantification of Liquefaction Potential

The quantification of liquefaction potential is carried out in accordance with Youd et al (2001). The estimation of two parameters are required to evaluate liquefaction potential:

A) CSR or Critical (or Cyclic) Stress Ratio –Demand on soil layers during the seismic event

B) CRR or Critical (or Cyclic) Resistance Ratio –Capacity of the soil to resist liquefaction

A Factor of Safety, FOS (= CRR/CSR) of greater than 1 is usually associated with non-liquefiable soil. A higher FOS may be warranted if uncertainty exists about the quality of data. The Indian Code IS 1893 Part 1 (2016) recommends a value of 1.2.

Sample calculations in accordance with RDSO Guidelines (2015) of a real-life project by use of worksheets are shown in Tandon et al (2015).

The evaluation is done in the following steps.

4.1 Determination of Design Ground Water level

Caltrans (2014) suggests that if the ground water table is at a level greater than 15m, the site should be considered non-liquefiable.

It is usual to decide this on the basis of water table in the area as per local records. Since the water table in the area at the time of carrying out subsurface investigations could be at a lower level, it is recommended that the values obtained (e.g. soil density) should be modified to take this into account while evaluating CSR. The ground water table affects the soil density and hence \( \sigma_v \), the Effective Vertical Stress in the evaluation of CSR. IS 1983 Part 1 (2016) and RDSO Guidelines (2015) are silent on the subject. Eurocode 5 (2004), suggests that free-field site conditions (ground surface elevation and water table elevation) prevailing during the life time of the structure should be adopted.

4.2 Making a realistic stratification of the soil from the subsurface investigation data

The soil characteristics such as SPT values, unit weight and fines content (passing IS standard sieve no: 75 microns in the India context) are identified.

4.3 Nomalisation of field SPT values

First we must normalise the field SPT values, N, to \( (N_{60}) \) and then to \( (N_{60})_{\text{eff}} \) for further processing, where,

\[
(N_{60}) = \text{SPT blow count of same soil for hammer with efficiency of 60%}.
\]

\[
(N_{60})_{\text{eff}} = \text{Value of (N_{60}) normalised for effective overburden pressure at ground level, i.e, atmospheric pressure, 98kPa}.
\]

The observed field test blow count values N need correction factor \( C_{60} \) to be applied to enable them to be converted to \( (N_{60})_{\text{eff}} \).

\[
(N_{60})_{\text{eff}} = C_{60} C_{\text{eff}}
\]

where

\[
C_{60} = \frac{P_r}{\sigma_v} = 9.79\left(\frac{1}{\sigma_v}\right)\sqrt{\frac{1}{\sigma_v}}
\]

\[
C_{\text{eff}} = C_{\text{ener}} C_{\text{hammer}} C_{\text{method}} C_{\text{length}} C_{\text{bore}}
\]

\[
C_{\text{ener}} = \text{Energy ratio correction}
\]

\[
C_{\text{hammer}} = \text{Hammer weight correction}
\]

\[
C_{\text{method}} = \text{Sampling method correction}
\]

\[
C_{\text{length}} = \text{Rod length correction}
\]

\[
C_{\text{bore}} = \text{Bore Hole diameter correction, and}
\]

\[
C_{\text{inst}} = \left(\frac{P_r}{\sigma_v}\right)^{1/2} = 9.79\left(\frac{1}{\sigma_v}\right)^{1/2}
\]
where $P_a$ is the atmospheric pressure.

A total of six numbers of corrections are applied on observed $N$ value to arrive at $(N)_{oc}$, There is no provision in the Indian codes for these corrections based on SPT equipment and methods employed in the country. Brief specification of the SPT equipment and methods are available in IS 2131 (1981).

The Standard Specifications for the SPT equipment recommended in Youd et al. (2001) is as per ASTM D1586 (2011), which is summarised in Table 1, and which is also included in RDSO Guidelines (2015).

In IS 1893 Part 1 (2016) it has been suggested that if the SPT values have been conducted as per IS 2131 (1981), the value of $C_{wv}$ may be taken as 1.

The RDSO Guidelines (2015), however, suggest that in the absence of test specific energy measurement, the corrections, $C_{wv}$ should in fact be carried out.

### 4.4 Evaluation of Cyclic Stress Ratio (CSR)

$$\text{CSR} = 0.65 (a_{max}/g) (\sigma_v/\sigma_v) \, r_o$$

$r_o$ = Stress Reduction Factor which depends on depth below ground level.

$$a_{max}/g = \text{(Ratio of Peak Horizontal Ground Acceleration/acceleration due to gravity)}.$$ For Zone IV, for instance, $MCE = 0.24$, IS 1893 which is suggested for liquefaction in case PGA is not available. Some codes like AASHTO (2012) recommend a value corresponding to a 975 year return period (5% probability in 50 years)

$$\sigma_v/\sigma_v = \text{(Total vertical Stress/Effective vertical Stress)}$$ should be evaluated for all the potentially liquefiable layers in the substrate. It would vary from approximately 2 to 1 depending upon where Ground Water Table is considered. It would be equal to 2 for Ground Water Table at Ground Level and equal to 1 if Ground Water Table is considered at the level lower than that where liquefaction is to be determined.

In equation (5) the flexibility of the soil profile is accounted for by $r$, which can be calculated from equations (6) and (6a).

$$r_1 = 1 - 0.00765 \cdot z \text{ for } z < 9.15 \text{m}$$
$$r_2 = 1.174 - 0.0267 \cdot z \text{ for } 9.15 < z < 23 \text{m}$$

The depth $z$ below the ground surface should be measured up to the center of the concerned layer.

### 4.5 Making correction for fines

Seed and Idriss (1982), concluded that liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil.

In accordance with IS 1893 Part 1 (2016), fines are defined as percent by weight passing the IS Standard Sieve No. 75μ.

The corrections for fines content can be done following the equations developed by Idriss with the assistance of Seed for correction of $(N)_{oc}$ to an equivalent clean sand value, $(N)_{cocos}$:

$$\frac{(N)_{ocos} - \alpha + \beta (N)_{oc}}{(N)_{oc}} = \frac{x + y}{z}$$

where $(N)_{ocos}$ is the blow count corrected for fines content, $\alpha$ and $\beta$ are coefficients that depend on the fines content.

$$\frac{(N)_{ocos} - \alpha + \beta (N)_{oc}}{(N)_{oc}}$$

where $\alpha$ and $\beta$ = coefficients determined from the following relationships:

$$\alpha = 0 \text{ for } FC \leq 5\%$$
$$\alpha = \exp \left[1.76 - \frac{190}{FC}\right] \text{ for } 5\% < FC < 35\%$$
$$\alpha = 5.0 \text{ for } FC \geq 35\%$$

$$\beta = 1.0 \text{ for } FC \leq 5\%$$
$$\beta = 0.99 + \left(\frac{FC1.5}{100}\right) \text{ for } 5\% < FC < 35\%$$
$$\beta = 1.2 \text{ for } FC \geq 35\%$$

### 4.6 Calculation of Cyclic Resistance Ratio (CRR)$_{7.5}$

The database from sites where liquefaction effects were or were not observed, a base curve for "clean sand" for Magnitude 7.5 Earthquakes was arrived. The curve was approximated to equation (10)-credited to AF Rauch of the University of Texas (1998)-which can be used more conveniently in worksheets.

### 4.7 Calculation of the Magnitude Scaling Factor (MSF)

The next step is to calculate the CRR for the particular site by evaluating the Magnitude Scaling Factor (MSF) for the particular site.

The data available for various parts of India for past earthquakes has been plotted in IS 1893 Part 1 (2016) which is reproduced in Fig. 5. If site specific investigations have not been carried out, Fig. 5 can be used for determining the Earthquake Magnitude, $(M)_s$, applicable to the site.

$$CRR_{7.5} = \frac{1}{34 - (N)_{ocos}} + \frac{(N)_{ocos}}{135}$$
$$+ \frac{50}{[10(N)_{ocos} + 45]} - \frac{1}{200}$$
4.8 Calculation of CRR

The CRR for the particular site is evaluated by multiplying CRR_{75} with MSF as shown in equation (12).

\[
\text{CRR} = \text{CRR}_{75} \times \text{MSF} \tag{12}
\]

4.9 Evaluation of Factor of Safety, FOS

The factor of safety with respect to potential of liquefaction is finally arrived at by using equation (13).

\[
\text{FOS} = \frac{\text{CRR}}{\text{CSR}} \tag{13}
\]

5. Conclusions

The paper highlights the methodology that should be employed in bridge design offices in the Indian context while determining the potential of liquefaction using SPT field tests. The frequently used codal provisions have been reviewed. Some differences between the two codes/guidelines, i.e., IS 1893 Part 1 (2016) and RDSO Guidelines (2015), have been highlighted. Some of the missing provisions in these codes have been discussed.

In many situations the liquefied layer is overlain by non liquefied strata. The bridge site may be located on sloping ground or near a water front, in which case Lateral Spreading would occur creating significant lateral forces on the piles. A simplified approach to account for the same has been identified. Both inertial effects and kinematic effects of liquefaction has been discussed in the paper.

Fig. 5 Epicenters of Past-Earthquakes (from IS 1893)

MSF is determined from equation (11).

\[
\text{MSF} = 10^{2.24/M_w^{2.56}} \tag{11}
\]

References


Idriss, I. M., and Boulanger, R. W. (2010), SPT-Based Liquefaction Triggering Procedures, Department of Civil & Environmental Engineering, Report no. Center for Geotechnical Modeling UCD/CGM-10/02, December


__, AASHTO LRFD Bridge Design Specifications (2012)
__, ASTM D1586 (2011), Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
__, IS 13920 (2016) Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces - Code of Practice
__, JRA (1996) Specifications for Highway Bridges in Japan
A large rail-cum-road double-deck multi-span steel truss bridge over the mighty river Brahmaputra is considered in this study. The bridge is in service for more than fifty years and moreover situated in the highest seismic zone of India. Thus, while monitoring the structural health of this bridge is an important issue, the study of modal parameters of this unique bridge structure is also a matter of great interest. The present work deals with operational modal analysis (OMA) of the bridge using measured acceleration responses under ambient vibration. Sensor locations are suitably identified for effectively extracting the major modes of this large bridge using limited sensors. Major modes are identified for this large bridge using standard identification techniques.

Keywords: Truss bridge, Operational modal analysis, Accelerometer, Ambient vibration.

1. Introduction

OMA using output-only response data is usually performed for large structures to identify modal parameters due to difficulties to perform experimental modal analysis (EMA). Modal parameters identified at different points of time are useful for structural health monitoring (SHM). Further, identified modal parameters are required to update a numerical model for better prediction of response and effective design of control devices. Various real life structures are found to be investigated for OMA e.g. Tsing Ma Bridge, Alfred Zampa Memorial Bridge, Gi-Lu bridge, FRP composite pedestrian truss bridge, North Grand Island Bridge, Cumberland River Bridge, Shanghai World Financial Center. Moreover, effects of temperature, traffic loading and wind speed on modal and dynamic behaviour of structure are also found to be analyzed.

Presently, many techniques exist for OMA in time domain. Natural excitation technique (NExT), a major development in time domain, establishes the similarity between cross-correlation function (CF) of ambient vibration responses measured at two locations and the
impulse response function (IRF) associated to those two locations. This helps to apply non-parametric techniques using CF instead of IRF which requires both input and output data for its computation. Eigen system realization algorithm (ERA)\textsuperscript{10}, one such frequently used technique developed based on minimum realization theory\textsuperscript{11}, identifies the state space matrices in multi-input multi-output (MIMO) framework. Modal parameters can be identified applying least-square based techniques e.g. least-squares complex exponential (LSCE) algorithm\textsuperscript{12} and poly reference least-squares complex exponential (PRCE) method\textsuperscript{13}. Another technique is Ibrahim time-domain (ITD)\textsuperscript{14} which identifies modal parameters using free decay responses. Free decay responses can be estimated from the ambient vibration responses using the random decrement technique\textsuperscript{15} and this makes it possible for the techniques like ITD to identify the modal parameters using ambient vibration data. Stochastic subspace identification (SSI) technique\textsuperscript{16}, a subspace based popular technique, identifies the structural system in the form of state space matrices directly from the ambient response data. Covariance-driven stochastic subspace identification (SSI-COR) technique\textsuperscript{17}, a modified version of SSI, identifies the modal parameters using singular value decomposition (SVD) of block Toeplitz matrix based on covariance of measured data. In many cases, total degrees of freedom (DOFs) may be divided into several set-ups using overlapping reference locations and data is acquired separately for different set-ups. Peeters and De Roeck\textsuperscript{17} presented a novel approach as SSI/ref to consider the data from multiple set-ups in the identification phase to estimate the global mode shape directly. OMA can also be carried out using auto regressive (AR) as well as auto regressive moving average (ARMA) based techniques\textsuperscript{18–21}.

The simplest method in frequency domain is the peak-picking (PP) method\textsuperscript{22} which works with assumptions of well separated frequencies and relatively low damping. To avoid such limitations, Brinckeret al.\textsuperscript{23} presented the frequency domain decomposition (FDD) technique based on SVD of power spectral density (PSD) matrix. First singular vector becomes an estimate of mode shape associated to a modal frequency and nearby PSD function helps to identify natural frequency and damping ratio. An improved version of FDD, Frequency-spatial domain decomposition (FSDD)\textsuperscript{24}, identifies natural frequency and damping ratio using curve fitting with enhanced PSD, a similar entity as enhanced FRF\textsuperscript{25}. Further, a maximum likelihood (ML) identification technique\textsuperscript{26} was proposed for modal identification. Peeterset al.\textsuperscript{27} presented poly-reference least-squares complex frequency (Poly MAX) technique based on stabilization diagram and least-squares frequency-domain (LSFD) methodology.

This paper deals with OMA of a rail-cum-road double-deck multi-span truss bridge, popularly known as Saraighat Bridge and built over the mighty river Brahmaputra, which is located in highest seismic zone of India. Acquisition of acceleration response with limited numbers of sensors at suitable locations is performed such that modal information of this large bridge structure may be effectively extracted. Both time and frequency domain techniques (NExT-ERA, SSI, FSDD) are employed.

2. Ambient Vibration Measurements

The Saraighat Bridge consists often main and two approach spans, which are all simply supported. The length of each of the main spans and approach spans are 118.72 m and 31.4 m respectively. A photographic view of the Saraighat Bridge is shown in Fig. 1. This rail-cum-road bridge carries rail and road traffic along lower and upper deck levels respectively. The height of truss is 18 m. Geometry as well as sectional properties of the bridge elements in each span are similar and hence, only one main span is considered for modal identification in this study.

Fig. 1 A photographic view of the Saraighat Bridge

Suitable placement of sensors can facilitate efficient identification of modal parameters. Locations of sensors are selected based on both qualitative and quantitative analysis as mentioned in the following steps:

(a) Usually, it is observed that deck-level plane (horizontal) is given importance in instrumentation of any bridge as it is easier to interpret any associated mode shapes for the horizontal, vertical and torsional behaviour of a bridge structure. In view of this, instrumentation is considered to be carried out along a deck level plane.
(b) Now, which deck-level is to be considered—rail level deck or road level deck? In the literatures regarding optimal sensor placement, it is observed that effective independence (EI) technique, system norm based modal-measures such as $H_2$, $H_\infty$, Hankel norms, modal measure in terms of modal contribution in output energy (MCOE) advocate for higher information/presence of a mode along DOF locations with higher modal deformation. Numerical model of the Saraighat Bridge, based on SAP2000 (V14.2.2), clearly demonstrates higher modal deformation along the rail-level plane associated to the prominent transverse and vertical modes as shown in Fig. 2. Therefore, the rail-level plane is considered as suitable location for sensor placement for the bridge under consideration.

Fig. 2 Significant eigen mode shapes along (a) transverse and (b) vertical directions
First three modes (transverse, vertical or torsional) are mainly targeted for identification. With the assumption of ideal simply supported behaviour, theoretical requirements of sensors locations are: (i) at $l/2$ for $1^{st}$ mode (ii) at $l/4$, $3l/4$ for $2^{nd}$ mode (iii) at $l/6$, $l/2$, $5l/6$ for $3^{rd}$ mode – collectively at $l/6$, $l/4$, $l/2$, $3l/4$, $5l/6$. Sensor location exercises are carried out taking into account $1^{st}$, $2^{nd}$ and $3^{rd}$ transverse and vertical analytical modes by employing both the EI approach and modal approach. Identified locations are observed to match fairly well with the theoretical locations. According to the positions of connections between longitudinal cord members and cross girders (potential sensor locations), possible locations are available at $l/16$, $2l/16$, $3l/16$, … $15l/16$. Required locations, except $l/6$ and $5l/6$, agree with these possible locations. Location $l/6$ falls between $2l/16$ and $3l/6$, while location $5l/6$ falls between $13l/16$ and $14l/16$. Finally, locations are considered at $2l/16$, $4l/16$, $6l/16$, $8l/16$, $10l/16$, $12l/16$, $14l/16$. Although, minor adjustment is considered for the required locations $l/6$ and $5l/6$, such considered locations however are supposed to capture the mode shapes with spatial uniformity. Moreover, the locations $6l/16$ and $10l/16$ are intended to improve the spatial resolution special for the most important $1^{st}$ mode (transverse, vertical or torsional).

Dense sensor networks are generally preferred for better spatial resolution of mode shapes, though limitation of availability of sensors becomes a common issue. With the employment of 15 uniaxial and 4 tri-axial accelerometers, the sensor-arrangement for the actual field study is considered as shown in Fig. 3. Based on the numerical mode shapes, it is observed that transverse (along $Y$) modal deformations for any two locations with similar $X$, $Z$ coordinates (e.g. $9L$ and $9R$ as in Fig. 3) become nearly equal. In view of this observation, respective locations of $3R$, $5R$, $15R$ and $3L$, $5L$, $15L$ are assumed to have same modal deformations in transverse ($Y$) direction and transverse measurements are taken only along left side of the deck. Measurements at the location $10L$ (along $Y$ and $Z$) are employed as reference channels which are useful in NExT to estimate cross-PSD functions only. Thus, a total of 27 channels are used for measuring data. Acceleration responses are measured using uniaxial, tri-axial force balance accelerometers (Model: EpiSensor ES-U2 and EpiSensor ES-T; make: Kinemetrics Inc., USA) and a 48 channel dynamic data acquisition system (model: MGChl; make: HBM GmbH, Germany). Acceleration and frequency measurement ranges (maximum) of ES-U2 and ES-T are ±4g and 0–200 Hz respectively. In this present work, data are recorded with a sampling rate of 100 Hz and resolution of 20 bit. The acceleration range for the accelerometers is adopted at quite lower range (±2g) to increase quality of the recorded time series.

3. Application of the Identification Techniques

The modal parameters are identified using three commonly used techniques: NExT-ERA, SSI and FSDD. Fifty five data-sets are selected out of total acquired ambient response data for performing OMA. Modal identifications are carried out individually for all these data-sets to have statistical inference. Duration of each data-set is around 250 seconds. Response-data with such duration are supposed to accommodate responses driven by excitations of wide frequency-range. Further increase of duration is not much found to increase performance. Data is processed at 12 Hz using a Butterworth filter of $5^{th}$ order and thereafter data is re-sampled at 24 Hz. Thus, requirement of anti-aliasing at 50 Hz is also fulfilled.
In the application of NExT-ERA, data from two reference channels are used. Such multiple-reference based NExT-ERA is also termed as MNExT-ERA, which helps to improve the performance in modal identification. Impulse response is estimated using inverse Fourier transform of cross-PSD functions. In computation of the cross-PSD, Hanning windows with 50% overlap are used to reduce effects of spectral leakage. Suitable size of Hankel matrix is investigated using Hankel matrices having various ratios of row-numbers and column-numbers considering a reasonably wide range from 0.1 to 10.

Next, in the application of SSI, size of the output block Hankel matrix is considered as an important parameter, which usually affects the performance in modal identification. Six cases of number of block-rows (i) are considered as {10, 25, 50, 75, 100, 125}, spanning from lower number to a reasonably large number of block-rows. In all these cases, total data points (n) having a maximum possible column number (j = n – 2i + 1) is used to construct the Hankel matrix. During implementation of FSDD, auto or cross PSD functions are computed using Hanning windows with 50% overlap to form the PSD matrix. A peak in the singular value plots of the PSD matrix indicates presence of a possible mode. In the present work, frequencies identified using NExl-ERA and SSI are also considered along with the frequencies corresponding to the observed peaks as possible modal frequencies. The first singular vectors of the output PSD matrices in the close-vicinity of a modal frequency are supposed to show higher MAC among them. This feature helps to identify final modal frequencies out of all the possible modal frequencies. Subsequently, all modal parameters are evaluated associated with these finally selected frequencies.

4. Identified Modal Parameters: Central Tendency and Dispersion

Modal parameters identified based on all the 55 data-sets employing all the techniques are taken into account for statistical characterization. This approach can ensure better acceptability in view of the probable presence of measurement noise as well as leakages in signal processing and the fact that the measured acceleration responses due to ambient vibration may differ from theoretical assumption of white noise induced vibration. Statistical analysis includes: (a) histogram observation on natural frequencies and damping ratios (b) central tendencies of natural frequencies, damping ratios and mode shapes (c) measure of dispersion and confidence intervals (CIs) of natural frequencies and damping ratios.

It may be observed from Table 1 that mean-values for modal frequencies identified using the three different techniques are fairly close to each other with low standard deviations as well as narrow confidence bounds. However, damping ratios identified using these three techniques show lesser agreement to each other as compared to natural frequency. Damping ratios estimated with FSDD are noticed to be comparatively lower than those estimated using NExl-ERA or SSI. Identified damping ratios also show higher dispersion in general as compared to the case of natural frequency. Mode shapes identified from different data-sets for a particular mode are averaged (after conversion from complex to real) to compute the mean mode shape. It may be observed from Table 2 that MAC values are quite high (a MAC above 0.8 may be considered as good for the considered large rail-cum-road truss bridge) except the case of 3rd mode. The 3rd mode shape identified using FSDD is somewhat different compared to the similar mode obtained through other two methods, though the shape apparently represents 2nd transverse mode shape very well.

5. Conclusions

OMA is carried out for a unique large rail-cum-road double-deck multi-span steel truss bridge employing three popular techniques both in time and frequency domain using measured acceleration responses. Limited numbers of sensors are appropriately used for effective identification of significant modal parameters. These are identified in a statistical manner based on multiple data-sets. Following concluding remarks are made based on the present study:

(a) Appropriate use of limited numbers of accelerometers for acquisition of ambient vibration response for the large bridge has yielded consistent modal parameters. Major transverse, vertical and torsional modes could be identified for the bridge.

(b) All the three identification techniques are observed to be consistent in OMA for this large bridge structure.

(c) A statistical approach in identification of modal parameters using multiple techniques both in time and frequency domains can ensure higher reliability in view of likely presence of measurement noise, leakage in signal processing and deviations in theoretical requirement of field ambient vibration being induced from white noise excitations.
### Table 1. Mean and 95% confidence bounds of identified natural frequencies and damping ratios

<table>
<thead>
<tr>
<th>Mode No.</th>
<th>Technique</th>
<th>Frequency (Hz)</th>
<th>Damping ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Mean</td>
<td>SD</td>
</tr>
<tr>
<td>1</td>
<td>NExT-ERA</td>
<td>0.9101</td>
<td>0.0189</td>
</tr>
<tr>
<td></td>
<td>SSI</td>
<td>0.9119</td>
<td>0.0094</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>0.9061</td>
<td>0.0146</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>1.8795</td>
<td>0.0549</td>
</tr>
<tr>
<td>2</td>
<td>SSI</td>
<td>1.8624</td>
<td>0.0529</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>1.8609</td>
<td>0.0341</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>2.1120</td>
<td>0.0033</td>
</tr>
<tr>
<td>3</td>
<td>SSI</td>
<td>2.1550</td>
<td>0.0492</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>2.1586</td>
<td>0.0757</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>2.9593</td>
<td>0.0204</td>
</tr>
<tr>
<td>4</td>
<td>SSI</td>
<td>2.9626</td>
<td>0.0157</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>2.9628</td>
<td>0.0201</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>4.4150</td>
<td>0.0535</td>
</tr>
<tr>
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<td>SSI</td>
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<td>0.0601</td>
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<tr>
<td></td>
<td>FSDD</td>
<td>4.4068</td>
<td>0.0489</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>5.1254</td>
<td>0.0094</td>
</tr>
<tr>
<td>6</td>
<td>SSI</td>
<td>5.1501</td>
<td>0.0288</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>5.1643</td>
<td>0.0347</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>6.3400</td>
<td>0.0436</td>
</tr>
<tr>
<td>7</td>
<td>SSI</td>
<td>6.3783</td>
<td>0.0720</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>6.3362</td>
<td>0.0490</td>
</tr>
<tr>
<td></td>
<td>NExT-ERA</td>
<td>6.9335</td>
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</tr>
<tr>
<td>8</td>
<td>SSI</td>
<td>6.9285</td>
<td>0.0482</td>
</tr>
<tr>
<td></td>
<td>FSDD</td>
<td>6.9729</td>
<td>0.0270</td>
</tr>
</tbody>
</table>

### Table 2. MAC values between identified mean mode shapes using three techniques.

<table>
<thead>
<tr>
<th>Mode No.</th>
<th>NExT-ERA/SSI</th>
<th>NExT-ERA/FSDD</th>
<th>SSI/FSDD</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.9833</td>
<td>0.9826</td>
<td>0.9915</td>
</tr>
<tr>
<td>2</td>
<td>0.9986</td>
<td>0.9992</td>
<td>0.9994</td>
</tr>
<tr>
<td>3</td>
<td>0.9726</td>
<td>0.6623</td>
<td>0.7341</td>
</tr>
<tr>
<td>4</td>
<td>0.9912</td>
<td>0.9779</td>
<td>0.9944</td>
</tr>
<tr>
<td>5</td>
<td>0.9964</td>
<td>0.9952</td>
<td>0.9925</td>
</tr>
<tr>
<td>6</td>
<td>0.9308</td>
<td>0.9238</td>
<td>0.9088</td>
</tr>
<tr>
<td>7</td>
<td>0.8879</td>
<td>0.9380</td>
<td>0.9103</td>
</tr>
<tr>
<td>8</td>
<td>0.9871</td>
<td>0.9930</td>
<td>0.9934</td>
</tr>
</tbody>
</table>
References


Challenges Faced (then And Now) during Kolkata Metro Construction – A Study

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1.0 INTRODUCTION

Kolkata, the city of joy, has its own glamorous history, which amazes just in its own perfect way starting with "what Bengal thinks today, India thinks tomorrow." Among all the historic possessions, this city has another first in its balance sheet - underground Kolkata Metro Rail. In the last twenty years, other cities in India have adopted the metro system. Currently Delhi, Bangalore and Chennai have underground stations in operation. Design & construction are in progress for Ahmedabad, Lucknow, Mumbai, Delhi, and Chennai to name a few.

The city of Kolkata lies in Gangetic West Bengal – about 200 km north of the Bay of Bengal. Kolkata Metropolitan District (CMD) with an area of 1250 sq.km and a population of more than 10 million has seen growth along the banks of Hooghly River which separates it from the neighboring city of Howrah. Kolkata is the major centre of industrial and commercial activity in Eastern India and the growth of population in recent years has put tremendous pressure on the very system it supports & sustains - the transport system. Between 1901 and 1951, 59% of the growth in the Kolkata urban area was in the central city alone but over the past two decades, the central city's growth has been minimal, adding 87,000 people from 1991 to 2011, while the suburbs added more than 3 million new residents. This intensifies the pattern of the last half-century where most growth clustered close to the city core like a number of major urban areas around the world.

The Calcutta (Kolkata) soil forms part of the Bengal basin which was the site of continuous deposition from the Cretaceous period on a geologic age profile. Later tertiary sediments were
deposited mostly by the Ajay & Damodar river systems on the west and from the Assam plateau on the east. The top 100m of the sediments are of recent origin consisting of successive layers of soft to firm silty clay/clayey silt followed by layers of dense sand/hard clay giving high end resistance (N* value) to load bearing piles as shown in Table 1. The terrain of Kolkata is almost flat with a general elevation of about 5-6m above MSL. The ground is higher near the river banks on the west but slopes down to the low lying marshy areas in the east and these areas often get flooded during monsoon rains.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Depth [m]</th>
<th>Description</th>
<th>Soil parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>0-2</td>
<td>Fill/Made ground</td>
<td>N=5.1<em>10^3,γ=18KN/m³, LL=50%,PL=30%,w=40%,Sand=5%,Silt=60%,Clay=35%,C</em>=25kPa,k=3x10^-6 cm/s</td>
</tr>
<tr>
<td>II</td>
<td>15</td>
<td>Soft grey/dark grey silty clay/clayey silt with decomposed wood</td>
<td>N=20,γ=19KN/m³, LL=50%,PL=22%,w=27%,Sand=6%,Silt=60%,Clay=34%,C*=50kPa,k=3x10^-7 cm/s</td>
</tr>
<tr>
<td>III</td>
<td>20</td>
<td>Firm bluish grey silty clay/clayey silt with calcareous nodules</td>
<td>N=45,γ=20KN/m³,k=2x10^-6 cm/s</td>
</tr>
<tr>
<td>IIIA</td>
<td>25</td>
<td>Medium/dense grey silty sand/sandy silt</td>
<td>N=45,γ=20KN/m³,k=2x10^-6 cm/s</td>
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<tr>
<td>IV</td>
<td>40</td>
<td>Brownish grey silty clay/clayey silt</td>
<td>N=35,γ=20KN/m³,LL=55%,PL=20%,w=25%,Sand=8%,Silt=65%,Clay=27%,C*=150kPa,k=2x10^-7 cm/s</td>
</tr>
<tr>
<td>V</td>
<td>45</td>
<td>Dense yellowish</td>
<td>N=50,γ=20KN/m³,k=2x10^-6 cm/s</td>
</tr>
</tbody>
</table>

Table 1: Generalised Subsoil in Kolkata area

2.0 BRIEF DISCUSSION

There has been a growing interest among policy makers about the relevance of rail-based systems, to address the mobility needs of the ever expanding population in Kolkata and with the meager network of road which covers no more than 10% of the land area, the situation is more chaotic. Kolkata (then Calcutta) Metro came under consideration in 1949 when French experts conducted a survey regarding which method of construction was to be followed here.

It was finally decided in 1972 following recommendation by Soviet Union Consultants that 'Cut & Cover' tunneling method aided by slurry wall construction was to be used mainly due to the fact that India's lack, at that time, of ability to handle soft ground shield tunneling under compressed air. But, when Calcutta Metro Project reached the 'take-off' stage in 1977 Metro decided to adopt both 'Shield Tunneling' & 'Cut & Cover' methods under built-up areas in Kolkata's northern sector containing underneath buried sewer lines, water mains, electrical cables, telephone cables, tram lines, overlying traffic, thickly populated old buildings, tidal circular canals, etc. For this, two specialized technologies had to be imported - the first one by M/s NIKEX Hungarian Co., Budapest which dealt with the construction of tunnels by shield tunneling method including excavation of tunnel under compressed air, erection of cast iron or precast concrete lining, grouting of annular space around lining by specialist Hungarian Technology & caulking of joints. The Second one concerning specialized technology for manufacture of precast steam cured concrete lining segments of thickness 200mm that is same as cast iron lining so that either type of lining could be used without any modification of shield. So, steam curing technology had to be adopted so that 90% of 28 days strength could be gained within 24hrs & ultimately progress of tunneling could be effected. Isothermic curing by means of saturated steam was done through a steam curing chamber made out of steel plates properly lined inside with fire bricks.

The north-south metro from Dum Dum to Tollygunj had been built almost entirely by the 'Cut & Cover' method except for a 1.09km stretch in the northern end which was built by Shield Tunneling with compressed air and air locks, because the alignment had to cross a railway yard near Belgachia station. The construction took a long time because the alignment was
to go through a busy urban area and underground construction technology was not as advanced as it is today. Operations commenced partly in 1985 and progressively extended to Tollygunj.

However, with the advent of latest mechanized tunneling (like Tunnel Boring Machine) with earth pressure balanced machines fitted with cutter head, screw conveyor, erector, belt conveyor, additive/grout injection system, tunnel guidance system supplemented by various backup systems such as segment transport device, earth transport device & alike various sequencing of tunnel operations unlike that of then methodology of 1970/80's has been made possible giving rise to many fold increase in the rate of progress to the extent of a month's progress of the then tunnel shield machines being achieved in a single day as seen in the east-west metro extending from Howrah Maidan to Salt Lake, which starts with the Cut & Cover construction for Howrah Maidan on the western side of the river & then ends on to the Howrah Station on the banks of the river. In between, Tunnel Boring Method (TBM) takes over. It goes under the river bed till it reaches a ventilation shaft on the eastern side of the Ganga. Thereafter, TBM follows all the underground alignment & continues up to Central station excepting the stations like Mahakaran, Esplanade, Swabhumi, Phoolbagan, Sealdah where Cut & Cover method is used. Further east the East-West Metro takes the overhead route through Salt Lake & terminates at Sector V. There again it intersects with the 32km long Garia-Airport Metro which follows the eastern periphery on viaduct. Side by side the Joka-BBD Bag Metro is being built overhead to meet the central Metro alignment at Esplanade. The rest of the Metro will be all overhead in the northern fringe of the city.

2.1 CUT AND COVER CONSTRUCTION METHOD

The design of cut & cover construction is still an empirical science and no established analytical procedure is as yet, available to include the effect of all the parameters to be considered for design. There are, in general, 4 possible modes of failure of braced excavation, viz.

i) Excessive movement of wall
ii) Yielding of supporting struts
iii) Bottom heave in cohesive soils
iv) Piping in granular soils/bursting in cohesive soils

Although the overall stability of braced cuts in soft ground doesn't depend to a large extent on the number & spacing of struts or anchors, they very much influence on the pattern of ground to be expected in a given situation. The depth of sheet pile/diaphragm wall determines the stability of the system & the ground movement associated with it. The strut loads were evaluated from the apparent earth pressure on braced cuts and also the ground settlement that occurred during excavation & strutting was proposed by Peck (1969). The Calcutta Metro in general had ground settlement around 2% of the depth of cut or 200-250mm for a depth of 12m. Monitoring were made on brick masonry buildings supported on shallow spread footings by observing the settlement on the plinth/floor level of the buildings as well as the tilt of the buildings. No major collapse had occurred although some buildings had undergone significant distress and required major repair and renovation which included the Statesman House & other heritage buildings on the Chowringhee Road.
2.2 SHIELD TUNNELING METHOD

At those times when Codes or Standards were not available, it is an international experience that one-cell precast reinforced concrete tunnel lining, constructed by shield method, under the ground water table is only satisfactory if the crack or damage of the segment is avoided, when the shield is thrust forward and the gaps between the segments have water tight seals, retaining their property for useful life of tunnel lining itself. Here heat/cold resistant asphalt coating with specified structural viscosity was applied to outside & radial face of segments to ensure distribution of forces transmitted by shield peripheral jacks during forward thrust and render the lining water and corrosion proof. The main role of the mineral grain structure of asphalt is that it allows the asphalt to flow and spread only to a certain thickness at which asphalt coating becomes load bearing.

Soil through which the tunnel alignment passed was of soft nature containing water & hence it was decided to drive the tunnel with the help of compressed air. The pressure was decided to be 1 Kg/cm² above normal to prevent seepage water coming into the tunnel.

Two shield complexes were brought from USSR which was made flame proof suiting tropical condition. In addition, 4 muck loaders with electric motor drive were also imported from erstwhile USSR. All the equipments of the shield complex were assembled & erected by Indian workers with the guidance from USSR experts. During those days of 70s/80s, productivity of 1m/day/shield was considered appropriate and the shield tunneling between Chitpur Yard & Shyambazar for a length of 780m was planned to be completed within a span of 4 long years, starting from 1979 onwards.

2.3 TUNNEL BORING METHOD & ITS IMPACT ASSESSMENT ON THE STRUCTURES

When a continuous ground is excavated by TBM, it's in situ stresses redistributes along the periphery experiencing a reduction on radial stresses & an increment on tangential ones causing radial strains with deformation known as convergences. In tunneling a support/lining is placed to resist the % of stresses which the ground cannot resist but there is a gap of time & space in between the moment when the tunnel is excavated & the lining/support is placed. During this mentioned gap, the ground is free to deform & 'volume loss' will take place in this period of time. The Kolkata Metro tunneling impact assessment was done for all the structures falling on the tunnels influence area. In that area shallow structures foundations, heritage & old dilapidated buildings, as well as deep structures foundations, flyovers & other buildings were assessed. The methodology applied for shallow structures is based on considering the buildings as a thick Timoshenko Beam [J.B. Burland et al.(2002) & Marco D. Boscardin et. Al(1989)] fixed or simply supported at both ends and applying a vertical displacement equal to the ground settlement in one of them or a punctual load will be applied to the centre. This analytical approach allows for calculating the shear & bending stresses (strains) on the building & checking of building damage as per Table 2.

<table>
<thead>
<tr>
<th>Category of Damage</th>
<th>Normal Degree of Severity</th>
<th>Limiting Tensile Strain ($\varepsilon_{lim}$)(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>0-0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very Slight</td>
<td>0.05-0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0.075-0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>0.15-0.3</td>
</tr>
<tr>
<td>4</td>
<td>Severe to very Severe</td>
<td>&gt;0.3</td>
</tr>
</tbody>
</table>

Table2: Damage Chart based on maximum strain

For Deep Foundation like Piles, tunneling will impact in two different ways; one will be structurally impact increasing bending moments & axial forces and the other will be affecting the bearing capacity of the pile through the ground friction reduction. The impact will depend on the position of the pile w.r.to tunnel centre, when the tunnel passes beside the pile, it will deform the ground creating bending moments on the piles because of the imposed displacements. Also a reduction in friction because of horizontal stresses relaxation will take place which may end up in settlements in piles.

<table>
<thead>
<tr>
<th>Sl No.</th>
<th>Structures</th>
<th>Construction type</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Colvin Court Building(G+3)</td>
<td>1920's Load Bearing Brick Masonry</td>
<td>i) Max. Settlement=70mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ii) Vol. Loss=3.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>iii) Smm cracks</td>
</tr>
<tr>
<td>Sl No.</td>
<td>Structures</td>
<td>Construction type</td>
<td>Damage</td>
</tr>
<tr>
<td>-------</td>
<td>------------------------------------------------</td>
<td>-----------------------------------------------------</td>
<td>----------------------------------------------------------------------</td>
</tr>
<tr>
<td>2</td>
<td>Bankim Setu Bridge</td>
<td>RCC</td>
<td>i) Max. Settlement = 15mm (predicted); Max. Settlement=5mm(actual)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ii) Vol. Loss=1.5%</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>iii) No cracks</td>
</tr>
<tr>
<td>3</td>
<td>DRM Building (G+2)</td>
<td>Load Bearing Brick Masonry</td>
<td>Max. Settlement = 3.5mm (actual) though predicted was high</td>
</tr>
<tr>
<td>4</td>
<td>Brabourne Road Flyover</td>
<td>Composite structure- Steel &amp; RCC</td>
<td>i) Max. Settlement = 20mm (predicted); Max. Settlement=10mm(actual)</td>
</tr>
<tr>
<td>5</td>
<td>Dilapidated Buildings along Raja Woodmount St.</td>
<td>Brick Masonry</td>
<td>i) Max. Settlement = 20mm (predicted); Max. Settlement=7mm (actual)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>ii) Vol. Loss =1.5% (predicted); Vol. Loss = 0.7% (actual)</td>
</tr>
<tr>
<td>6</td>
<td>Writers Building &amp; St.Andrews Church</td>
<td>Brick Masonry</td>
<td>i) Max. Settlement = 22mm (predicted); Work on progress</td>
</tr>
</tbody>
</table>

### 3.0 ACKNOWLEDGEMENT

The author wishes to acknowledge the help extended by Consulting Engineers Association of India in organizing the Seminar on Kolkata East West Metro – Unique Project Challenges on 14th October, 2017 at Williamson Magor Hall, Bengal Chamber of Commerce & Industry, 6 N.S. Road, Kolkata-01 which was also supported by Kolkata Metro Rail Corporation Limited (KMRCL).

### 4.0 CONCLUSION

Though much literature is not available on construction technologies adopted during Kolkata Metro Rail construction this paper tries to do justice in highlighting the major challenges faced then & now. With the advent of latest mechanized tunneling like TBM, various sequence of tunnel operations unlike the then methodologies of 1970’s/80’s has been made possible to be concurrent & continuous giving rise to a very high increase in the rate of progress and nevertheless imparting safety, stability and yet contributing least to the environment carbon debits and health hazards.

### REFERENCES

2. Dr. Alvaro Casasus & Harvinder Rana, Associate Director & Design Engineer, AECOM, Gurgaon, India, ‘Impact of underground construction and TBM tunneling on surface Structures in Kolkata East-West Metro’, Seminar on Kolkata East West Metro, 14th October, 2017.
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