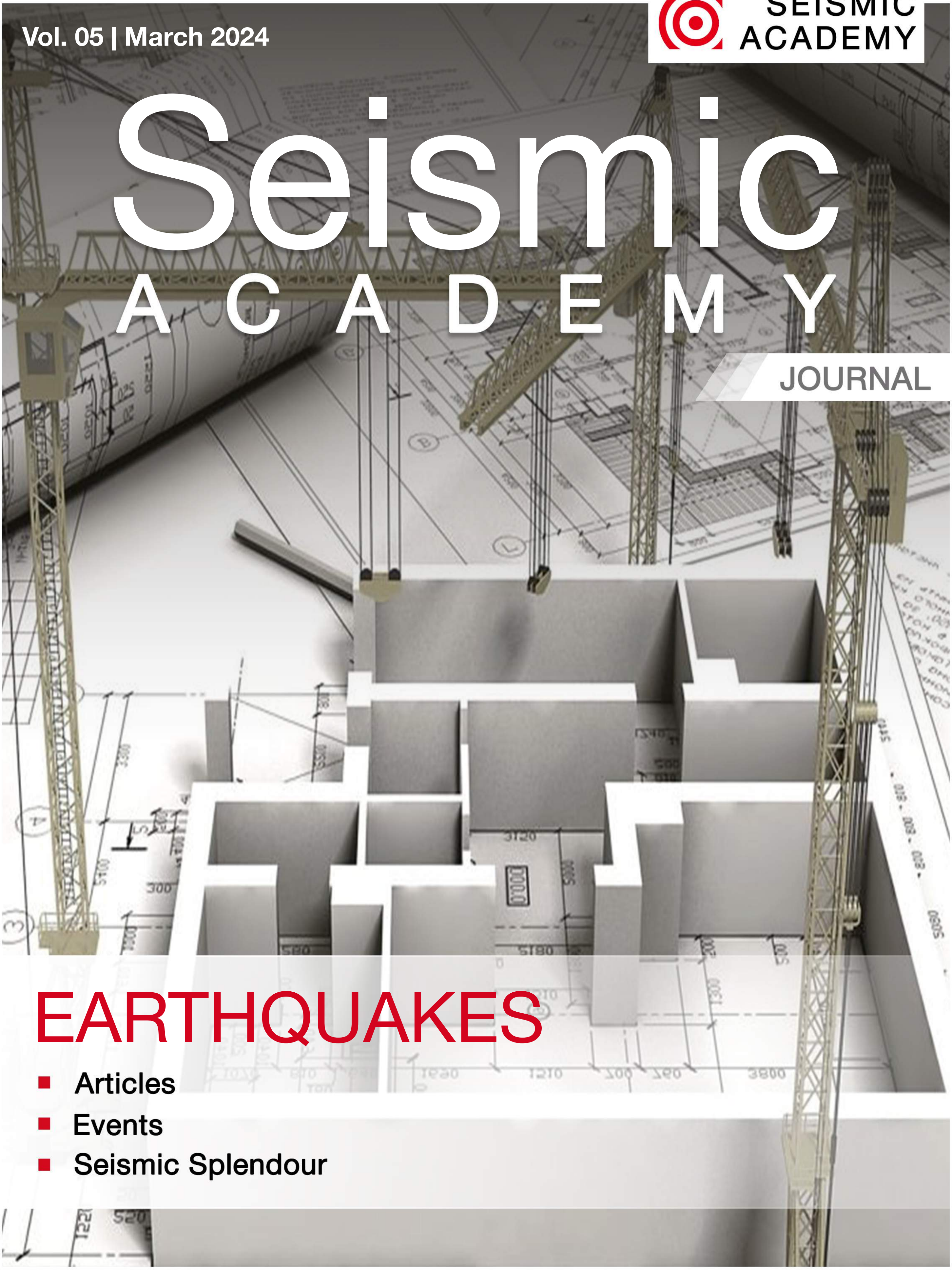
Vol. 05 | March 2024

SEISMIC ACADEMY

Seismic ACADEMY JOURNAL



EARTHQUAKES

- Articles
- Events
- Seismic Splendour

Seismic Academy Journal

An Initiative by HILTI
Building Benchmarks for Industry and Academia

A CE&CR Production
unmatched coverage

SEISMIC ACADEMY

A forum for professionals, academicians, authorities and industry experts to interact and disseminate knowledge on various aspects of earthquake engineering with different stakeholders, with an intent to increase awareness and develop their expertise on the subject.

OUR VISION

To make seismic academy as one source of information and use it for promotion of all seismic initiatives in our country.

OUR ADVISORY BOARD

A CE&CR PRODUCTION

IN THIS ISSUE - Total No. of Pages: 60 (Including Cover)

For further information, contact -

Shounak Mitra, Head - Codes & Approvals (Fastening), Hilti India Pvt. Ltd.
M: 9899732127 | E: support@theseismicacademy.com

SEISMIC ACADEMY

FROM THE DESK OF ADVISORY BOARD

Earthquakes cause devastation at places where the community is not adequately prepared. Particularly, the loss of life because of earthquake-induced damages to the built infrastructure is a matter of grave concern. One possible way through which lives could be saved during earthquakes is by giving an early warning so that people may find safe places to rescue - e.g., go to open ground or stay in tents temporarily - but evacuate the seismically vulnerable massive buildings that they stay in otherwise.

However, it is argued that time of occurrence of tectonic earthquakes is unpredictable. Though some research work is carried out by developing early warning system (EWS) for sending alerts or distress signals, the state-of-the-art remains at giving the warning hardly few seconds to a minute or two in advance. Albeit, considerable research efforts should have been geared towards developing sophisticated EWS so that the time to give warning signal could be increased. However, deterrent in pursuing this domain of research confidently is attributed to the fear of giving false signal(s), or more seriously, for not being able to give a warning signal at all before a major earthquake happens.

Nevertheless, in true scientific spirit, researchers should be encouraged to work on developing EWS to be able to give signals much in advance to the occurrence of an earthquake. Thereby, the common belief of unpredictability of occurrence of earthquakes can be shattered, and innovative EWS could be developed for the benefit of the civilization. Seismic Academy is advised to address this matter effectively and promote research activities in development of advanced earthquake early warning systems, which can be linked with smartphones so that eventually we are able to attain earthquake-safe society.

Prof. Dr Vasant Matsagar
Professor, Dogra Chair and Head,
Department of Civil Engineering,
Indian Institute of Technology
(IIT) Delhi,

“

In true scientific spirit, researchers should be encouraged to work on developing EWS to be able to give signals much in advance to the occurrence of an earthquake.

”

TABLE OF CONTENTS

Dr. Shailesh Kr. Agrawal
Executive Director
Building Materials & Technology Promotion Council (BMTPC)
(Ministry of Housing & Urban Affairs, Government of India)

“Resource optimization in building material sector is key to sustainable development.”

Seismic Academy: Walk us through your vision of Indian Infrastructure 2025 for the building material industry.

Dr. Shailesh Kr. Agrawal: In the wake of climate change & global commitments to bring down GHG emissions & march towards net zero sustainable development, it is high time, we take a leap from conventional construction practices & transit towards resource-efficient, climate-responsive, energy-efficient disaster-resilient materials, processes, systems & technologies. Cast-in-situ masonry/RCC construction using brick by brick/brick & sticks is being replaced globally by offsite construction wherein building elements can be produced under controlled conditions in a factory setup or casting yard and transported to the site for assembly & making the desired structure. These industrialized building systems are time-tested & proven around the globe & being practiced in the country also. The Indian construction industry is picking these practices slowly but surely. Therefore, Industrialized building systems are the future of Indian Infrastructure.

Also, to be developed nation by 2047, the rapid construction systems which reduce the construction time considerably are imperative & will replace the existing RCC framed construction. As regards materials, cement & steel are major ingredients of any construction but these materials are based on finite natural resources and therefore to be used optimally. Resource optimization in building material sector is key to sustainable development. Cement blended with supplementary cementitious materials (SCM) such as fly-ash, slag, GGBFS; materials based on renewable resources, waste, by-products; recycled/refurbished materials; green concrete, geo-polymer concrete; geo-polymer coarse aggregates; fly-ash to sand; nano concrete aggregates; artificial sintered aggregates; manufactured sand; slag sand; FRP/GFRP steel bars, fibres replacing rebars are few in the future list of materials. Smart materials, materials with less embodied energy & eco-labelling of materials is the vision for futuristic materials.

Seismic Academy: What all, in your opinion, have been paradigm shifting practices incorporated in India’s New Age construction processes?

Dr. Shailesh Kr. Agrawal: Govt. of India through Ministry of Housing & Urban Affairs conducted Global Housing Technology Challenge-India (GHTC-India) to transplant globally available proven best construction practices to India in 2019 & this challenge has been the trailblazer in triggering the technology transition in the construction sector. I must urge readers to go through our website https://ghtc-india.gov.in & know it all about the India’s New Age construction systems being promoted by us & concerted efforts are made to create enabling eco-systems to mainstream them. We have created a basket of 54 futuristic technologies for stake holders to pick & choose as per geo-climatic conditions & cost/time constraints. CPWD has issued SORs for most of them & also published a detailed circular on their applicability. However, for the sake of readers, let me explain you these systems briefly. The 54 technologies are divided into 6 broad categories namely

- Precast Concrete Construction System - 3D Precast volumetric: a system where 3D RCC modules/ PODs are cast & transported to the site for assembly
- Precast Concrete Construction System - Precast components assembled at site: Planar RCC building components such as walls, slabs, staircases, sun-shades, facades, beams, columns are cast offsite & assembled at site.

- Light Gauge Steel Structural System & Pre-engineered Steel Structural System: Steel frame comprising of hot rolled steel sections or cold-form(light-gauge) steel frames along with different infill options
- Prefabricated Sandwich Panel System: Dry wall construction replacing conventional masonry walls. These Sandwich panels comprise of lighter core material sandwiched between two outer wythes/sheathings & can be used for load-bearing/non-load bearing applications
- Monolithic Concrete Construction: Customized formwork systems which allow casting of walls & floors together thereby enabling robust monolithic construction. Also known as Aluminum form work systems. Some form works use steel also as form work material such as tunnel form work
- Stay-in-Place Formwork System: It is lost or sacrificial form work which is left in the structure to act as part of the structure. There are PVC wall forms which are prefinished walls & can be erected directly

Under PMAY-U, these six systems are being demonstrated by constructing six Light House Projects (LHPs) consisting of 1000+ houses each at six locations namely Chennai, Ranchi, Indore, Rajkot, Lucknow & Agartala. These LHPs are projected as live laboratories for stakeholders to learn & emulate. There are other recent developments such as 3D printing, cloud-based project monitoring, block-chain construction management, construction using robots. Construction 4.0 i.e. digital transformation in the sector is next major revolution.

"Embark on India's construction revolution with the GHTC-India by the Ministry of Housing & Urban Affairs. Explore 54 futuristic technologies for sustainable building solutions."

Seismic Academy: What are BMTPC's offerings in terms of skill development, disseminating material awareness and creating environment for innovative technologies to the construction industry?

Dr. Shailesh Kr. Agrawal: In order to have an integrated approach for comprehensive technical & financial evaluation of emerging and proven building materials & technologies, their standardisation, developing specifications and code of practices, evolving necessary tendering process, capacity building and creating appropriate delivery mechanism, Ministry of Housing & Urban Affairs, Government of India has set up a Technology Sub-Mission under PMAY-U with the Mission statement as Sustainable Technological Solutions for Faster & Cost Effective Construction of Houses suiting to Geo-Climatic and Hazard Conditions of the Country.

The Technology Sub-Mission facilitates (a) adoption of modern, innovative and green technologies and building material for faster and quality construction of houses (b) preparation and adoption of layout designs and building plans suitable for various geo-climatic zones (c) assisting States/Cities in deploying disaster resistant and environment friendly technologies.

BMTPC is mandated to identify, evaluate and promote emerging construction systems suiting to different geo-climatic conditions of the country, which are safe, sustainable and environment-friendly and ensure faster delivery of quality houses. The Government of India has authorized BMTPC to certify such new systems through Performance Appraisal Certification Scheme (PACS) (vide Gazette Notification No. I-16011/5/99 H-II Vol 49 dated 4th December, 1999). The third edition of Compendium of Prospective Emerging Technologies for Mass Housing has been published and can be downloaded from www.bmtpc.org.

In order to facilitate adoption of alternate and emerging technologies by the State Governments, Ministry of Housing & Urban Affairs has pursued CPWD, BIS and State departments to come out with notifications, Circulars, SORs, specifications etc. which will authorize State governments to use these new construction technologies in housing projects.

For better advocacy & wider dissemination & also to showcase the field application of innovations in the construction sector technologies, MoHUA has taken an initiative to construct Demonstration Housing Projects (DPHPs) across the country through BMTPC. These DHPs are also used to impart hands-on training to professionals & artisans during construction.

To build capacities of engineers & architects, MoHUA is also running an online Certificate Course on Use of Innovative Construction Technologies "NAVARITIH" in collaboration with BMTPC & School of Planning & Architecture, New Delhi (https://ict.bmtpc.org).

In order to catalyze the market for affordable housing, MoHUA has been encouraging State Governments and large public agencies like housing boards, railways, defence and public sector units to undertake construction of their projects using innovative technologies.

> **Seismic Academy:** Precast construction is gradually gaining prominence in India. How would you evaluate the performance of precast structures in the event of moderate to severe earthquakes. Given India as a country is fairly prone to these tremors?

Dr. Shailesh Kr. Agrawal: First of all, it is a misconception that Precast concrete construction will not perform better than conventional construction during shaking of ground. A few structures fell during earthquakes but since then we have acquired more know-how, learnt lessons & conducted experimental & analytical investigations to supplement performance based precast concrete construction. Nevertheless, we have not allowed precast concrete construction in Zone V in India which is the most severe earthquake zone. Also, full scale testing of protype structures using innovative technologies is being encouraged to study seismic behaviour of buildings being bult in Indian context & to gain further insights. BIS has also come out with brand new comprehensive codes on precast construction along with ICI precast handbook which help design & construct such buildings. CPWD has also recommended use of precast construction up to Zone IV.

> **"Full scale testing of protype structures using innovative technologies is being encouraged to study seismic behaviour of buildings being bult in Indian context & to gain further insights."**

> **Seismic Academy:** What specific projects or programs has BMTPC undertaken to introduce and implement advanced technologies aimed at improving earthquake resilience in the country?

Dr. Shailesh Kr. Agrawal: India has a history of disasters leading to irretrievable losses to lives and properties on account to its geological settings and distinct demography. Realizing the need, there have been concerted efforts made by Government of India to bring paradigm shift in its approach towards disaster risk reduction. The traditional 3 Rs (Rescue, Relief & Restoration) are now replaced by 3 Ps (Prevention, Preparedness & Proofing) and pro-active pre-disaster preventive measures are part and parcel of India's growth story. Some of the watershed moments in the annals of disaster management in India are enactment of Disaster Management Act, formulation of Disaster Management Policy and National Disaster Management Plan which are in line with UN resolutions, Hyogo framework (2005-15), Sendai framework (2015-2030) on natural disaster reduction and sustainable development goals. One of the major contributions of BMTPC i.e., bringing out Vulnerability Atlas of India which till date is the only document existing on damage risk to existing housing stock in India w.r.t. natural hazards e.g. earthquake, wind & cyclone and flood. The atlas was first published way back in 1997, and then in 2006, 2008 (CD form) based on 2001 Census data and then third edition of the Vulnerability Atlas of India was brought out in 2019 updated based on updated recent available data from IMD, Survey of

India, Geological Survey of India (GSI) and Census 2011. It includes hazard maps of earthquakes, wind/cyclones, floods, landslides, thunderstorms and vulnerability risk tables based on available latest data in order to help in enhancing preparedness of Governments and various other agencies in mitigating natural disasters. The Atlas is a useful tool not only for public but also for urban managers and National & State Authorities dealing with disaster mitigation and management.

Seismic retrofitting of existing vulnerable buildings is one of the most challenging tasks before the architects & structural engineering fraternity. A large number of existing buildings in earthquake prone areas over the world need seismic retrofitting due to various reasons & motivations, including codal modifications, deterioration of structures with age or change in use or modification of structure. Earthquake damaged buildings may also need retrofitting along with repair of damaged portion for reuse. Seismic retrofitting of existing stock is one of the most effective methods towards seismic risk reduction in future & to have safe & better habitat. In its efforts to demonstrate the retrofitting techniques for seismic strengthening, BMTPC undertook the retrofitting of few MCD school buildings & life line buildings so that the awareness could be generated among the stakeholders as well as various government agencies about the need and techniques of retrofitting. The experience on these buildings would help people at large and the policy makers in working towards reducing the vulnerability of lakhs of existing buildings through retrofitting of public and private buildings.

“Seismic retrofitting of existing stock is one of the most effective methods towards seismic risk reduction in future & to have safe & better habitat.”

Seismic Academy: Please throw some light on the regulatory policies which the government is planning to ensure sustainable construction practices, both in terms of structural resilience as well in terms of overall sustainability.

Dr. Shailesh Kr. Agrawal: India is witnessing rapid urbanisation with about 377 million people comprising 31.14% of the total population lived in urban areas (Census 2011). The urban population is projected to grow to about 600 million by 2031. While cities are engines of growth, they also contribute to more than 70% of India's greenhouse gas (GHG) emissions leading to extreme weather events.
India is witnessing perceptible increase in number as well as intensity of extreme weather events in recent times. India has unique geo-climatic and socio-economic conditions, and is vulnerable, in varying degrees, to rising sea levels, floods, droughts, cyclones, landslides, avalanches, storms, and heat waves. It is estimated that India will experience a decline of about 2-6% in its GDP under the carbon-intensive scenario by 2050, which could pose a serious threat to its development goals and investments. India is committed to reduce its emissions by 2030 up to 45% & become carbon neutral by 2070. Therefore, sustainable habitat is need of the hour.

National Mission on Sustainable Habitat 2021-2030 is in place. The excerpts from the mission document are reproduced here. Sustainable Habitat is an approach towards a balanced and sustainable development of the ecosystem of habitat which offers adequate shelter with basic services, infrastructure, livelihood opportunities along with environmental and socio-economic safety including equality, inclusiveness and disaster-resilience. It can be broadly divided into five areas, namely
(i) Energy and Green Building: It focuses on reducing the energy consumption for HVAC, etc. in India's real estate sector and shifting to cleaner renewable energy sources through adoption of green building technologies.
(ii) Urban Planning, Green Cover and Biodiversity: It lays emphasis on integrated urban and regional planning approaches to climate-sensitive development and preservation and rejuvenation of water bodies, green spaces, and eco-sensitive areas.
(iii) Mobility and Air Quality: focuses on inclusive and multi-modal mobility options in order to arrest the

rapid growth of private motor vehicles, which has led to traffic congestion and increasing air pollution levels in metro cities. (iv) Water Management: lays emphasis on augmenting existing water resources by adopting rain-water harvesting (RWH), rejuvenation of waterbodies, recycling/ reuse of treated sewage, water conservation, and promoting circular economy of water. and (v) Waste Management: focuses on the need for cities to prioritise actions for waste reduction and waste management, and promote waste-to-energy and waste-to-compost plants.

In 2024, we will also have Eco-Niwas Samhita (ENS) 2024 (Energy Conservation and Sustainable Building Code) for Residential, Commercial & office Buildings. Note, the word sustainable has been added in the erstwhile ECBC code of 2018.

As regards Disaster risk reduction, there have been several policy documents in forms of act, policy, plan, Techno-Legal Regime & building bye-laws. Ministry has published Model Building Bye-Laws-2016 (MBBL-2016) for the guidance of the State Governments, Urban Local Bodies, Urban Development Authorities. Building Bye-Laws are legal tools used to regulate design and construction aspects of buildings. They are mandatory in nature and serve to protect buildings against fire, earthquake, noise, structural failures and other hazards. As regards, safety & security, the chapter 6 of MBBL is on provisions for structural safety which includes Structural Safety, Disaster management as per late Prof. Arya Committee Report and BIS Codes including Structural Design Basis Report (SDBR) for various building types, seismic strengthening/retrofitting & prevention measures against Soft Storeys in multi-storeyed buildings and Proof Checking of Structural Design for buildings.

> **“India is committed to reduce its emissions by 2030 up to 45% & become carbon neutral by 2070. Therefore, sustainable habitat is need of the hour.”**

Seismic Academy: BMTPC’s collaboration with NIDM and NDMA for disaster management is widely known. How do you foresee the association with Seismic Academy also, in this regard to ensure we reach greater section of the fraternity? What is your recommendation for Seismic Academy going forward?

Dr. Shailesh Kr. Agrawal: In today context, when we have reached to a point where enough literature & data is available with regard to seismic design, construction & other related topics, it is time to reach out to the people & get it translated in the field in letter & spirit. Seismic Academy can play a crucial role in spreading awareness, building capacities, educating professionals & artisans & can take up demonstration projects to showcase cutting edge materials & technologies to enhance disaster resilience. Information, Education, and Communication (IEC) can play a pivotal role in empowering communities with knowledge and skills to improve resilience of the society. Seismic Academy can delve into the significance of IEC in community development, emphasizing its role in promoting awareness, fostering participation, and catalyzing sustainable change.

Seismic Academy: Any key message for the students and young practicing engineers?

Dr. Shailesh Kr. Agrawal: As per clarion call given by our Hon’ble PM, let us contribute towards nation’s growth & take our country towards developed nation by 2047 in a most positive & befitting manner to best of our capabilities. India is cruising to become $5 trillion economy & world’s third largest. The construction sector in India is emerging as third largest sector globally & reached $ 750 billion in value. It is therefore obligatory upon us to be Receptive, Innovative and Productive to foster sustainable growth and ensure better quality of living. For fellow citizens.

Training Program on

URBAN RISK MITIGATION – FOCUS ON SEISMIC SAFETY OF STRUCTURES ORGANISED

A 3-day workshop was organized by National Institute of Disaster Management (NIDM) in collaboration with Bureau of Indian Standards (BIS), Building Materials and Technology Promotions Council (BMTPC), Delhi Disaster Management Authority (DDMA) and Seismic Academy, initiative by Hilti on "Urban Risk Mitigation – Focus on Seismic Safety of Structures" from 13th March 2024 to 15th March 2024 at Rohini, New Delhi.

The workshop was attended by senior officers and representatives from more than 17 states and union territories and from Delhi Disaster Management Authority, West Bengal Disaster Management Authority, Bihar State Disaster Management Authority, Sikkim State Disaster Management Authority, Nagaland State Disaster Management Authority, Assam State Disaster Management Authority, UP State Disaster Management, State Irrigation Department - UP, PWD Uttarakhand, PWD Punjab, PWD Leh Ladakh, Planning & Development–Daman & Diu as well as people from MCD, Health Department & PWD – Delhi NCR.

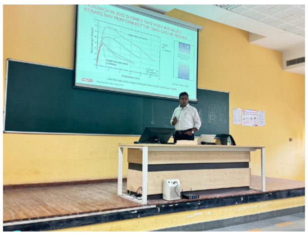
The workshop had clear direction setting in the inaugural session by Shri Rajendra Ratnoo (IAS), Executive Director, National Institute of Disaster Management, Dr. Shailesh Agrawal, Executive Director, Building Materials and Technology Promotion Council, Mr. Surendra Thakur, Joint Director, National Institute of Disaster Management, Dr. Garima Aggarwal, Senior Consultant, National Institute of Disaster Management and Mr. Shounak Mitra, Head-Codes and Approval, Hilti India Pvt. Ltd.

The three days were full of enriching sessions by the likes of Dr. Garima Agarwal, Dr. Neeraj Kumar (Faculty – Central University of Haryana), Mr. Jitendra Chaudhary (Member Secretary – Bureau of Indian Standards), Mr. Pradeep Garg (Superintending Engineer – CPWD), Dr. Shailesh Agarwal (BMTPC), Dr. Pratima Rani Bose (Associate Director – DDF Consultants), Prof. CVR Murty (Faculty – Indian Institute of Technology, Chennai), Mr. Anup Karanth (World Bank) and Mr. Shounak Mitra (Hilti India Pvt. Ltd.).

The workshop also had a dedicated visit to the structural health monitoring laboratory of CSIR – Central Road Research Institute (CRRI) as well as demonstration of right job site execution practices by Hilti.

To know more - https://theseismicacademy.com/workshop-detail/national-training-program-on-urban-risk-mitigation-focus-on-seismic-safety-of-structures

WORKSHOP ON
REPAIR, REHABILITATION AND RETROFITTING OF STRUCTURES



A half-day workshop on “Repair, Rehabilitation and Retrofitting of Structures” was conducted on 19th March 2024 at Birla Institute of Technology and Science (BITS) Pilani under the aegis of the Seismic Academy. Mr. Rohit Yadav, Managing Director, Texel Consulting Engineers shared his insights on different case studies on structural repair, rehabilitation and retrofitting followed by Shounak Mitra, Head – Codes & Approval, Hilti India Pvt. Ltd. highlighting the importance of concrete-to-concrete connection design for ensuring effective performance of the retrofit scheme. The session was attended by 45 higher degree students of BITS Pilani and other adjoining colleges and created lot of enthusiasm among the participants.

The workshop was conducted under the inspiration of Dr. Anupam Singhal, Head of Department – Civil Engineering, BITS Pilani and Dr. Muthu Kumar G – Assistant Professor - Civil Engineering, BITS Pilani

To know more, click -
https://theseismicacademy.com/workshop-detail/repair-rehabilitation-and-retrofitting-of-structures

NON-LINEAR TIME HISTORY ANALYSIS OF ELASTOMERIC BASE-ISOLATED BUILDING STRUCTURE

Om Gomase
Dy. General Manager (Design)
Epicon Consultant Pvt. Ltd.

The influence of isolator Characteristics on seismic response of multistory base isolated structure is investigated. The force deformation behavior of an isolator is modeled as non-linear hysteretic behavior for different time period with the effect of soil characteristics. Uniform Building Code (UBC-97) is widely used in design of base isolation systems which contains provision according for near fault effect. To assess the effectiveness of base isolation systems, a study was conducted on a four-story building designed in accordance with UBC-97 regulations, specifically for near fault earthquakes. The building is situated near an active fault line. The isolation system utilized in this structure consists of high damping rubber bearings. Design displacements were determined using UBC-97 parameters. The building was subjected to three different earthquake events: the 1979 El-Centro, 1995 Kobe, and 1994 Northridge earthquakes. A comparison was made between the fixed base and base isolated structure, with a focus on the variation of the time period to analyze the parametric changes in isolator characteristics.

Keywords: Base isolator, Effective Stiffness, Hysteresis, Post yield stiffness.

INTRODUCTION

One of the most widely implemented and accepted seismic protection systems is base isolation. Seismic base isolation is a technique that mitigates the effects of an earthquake by essentially isolating the structure and its contents from potentially dangerous ground motion, especially in the frequency range where the building is most affected. The goal is to simultaneously reduce inter-story drifts and floor accelerations to limit or avoid damage, not only to the structure but also its contents, in a cost-effective manner.

1. Horizontal flexibility to increase structural period and reduce spectral demands (except for very soft soil sites)
2. Energy dissipation (also known as damping) to reduce isolator displacements, and (3) sufficient stiffness at small displacements to provide adequate rigidity for service-level environmental loadings. The horizontal flexibility common to all practical isolation systems serves to uncouple the building from the effects of high frequency earthquake shaking typical of rock or firm soil sites, thus serving to deflect the earthquake energy and significantly reduce the magnitude of the resulting inertia forces in the building. Energy dissipation in an isolation system, in the form of either hysteretic or viscous damping, serves to reduce the displacement response of an isolation system generally resulting in more compact isolators.

Structural response and isolator displacement are two key parameters to decide the characteristics of an isolation system. In near-field area isolator displacement plays vital role in governing the design of an isolation system, as large isolator displacements leads failure of isolation system. To check isolator displacement, stiffness of isolation system is increased but such increase adversely affects the structural response, especially floor accelerations. Present study aimed to explore the role of increase of isolation stiffness on structural response of a building. Bi-linear isolation system is selected for the study.

The bilinear model, used to express the relation between the shear force and the lateral displacement, can be defined by three parameters: initial stiffness, post-yield stiffness, and characteristic strength. The characteristic strength, Q, is usually utilized to estimate the stability of hysteretic behavior when the bearing experiences many loading cycles. These three parameters properly reflect the mechanical properties of bearings and provide satisfactory estimations of a bearing's nonlinear behavior. The specific objectives of the study are:

1. to investigate the effects of increase of initial stiffness on structural response
2. to analyze the effect of isolation period on structural response
3. to investigate the effects of characteristic strength ratio of isolator on structural response.

MATHEMATICAL MODELING OF FIXED-BASE BUILDING STRUCTURE

Typical floor plan and elevation of base isolated 4 storey reinforced concrete structure building, which is used as the subject structure in this study as shown in Fig. 1 and Fig. 2 respectively. All columns are 30 x 55 cm. and beams are 40 x 50 cm with floor heights are 3.1 m. There are 3 bays of 5m in X-direction, 3 bays of 2m, 3m and 2m in Y-direction, i.e. plan dimensions are 15 m x 8 m. The total mass of the building is 1400 tons corresponding to the weight of W=14250 kN. All structural members are of concrete with Fck=20 N/mm^2 and Fy=415 N/mm^2. The fixed-base periods of superstructure in each direction are 0.75 seconds and the superstructure modal damping ratios are assumed to be constant for each mode as 5%.The superstructure is placed on an isolation system consisting of high-damping rubber bearings placed under each column. Since, it is considered that the weight is equally transfer to each bearing under the column. There exists a rigid slab at the base level that connects all isolation elements. The three-dimensional model of the base-isolated building and the non-linear time-history analyses are made using a well-known software program SAP2000 version (11).

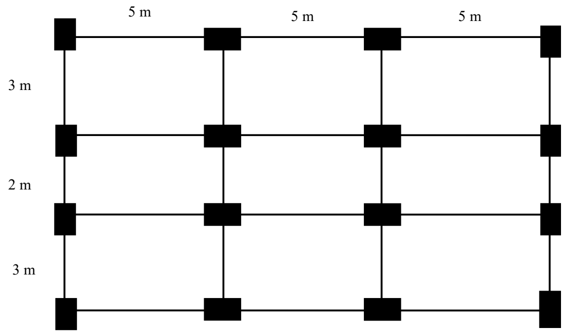


Fig. 1 (a) Typical floor plan

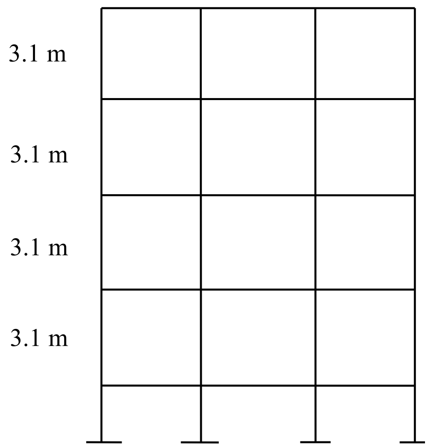


Fig. 1 (b) Elevation

The building is assume to be located in high seismicity region, i.e. Zone 4, and assigned a seismic zone factor Z=0.4 according to Table 16-I of the UBC-97. The actual time history data has been carried out specifying closest distance to a known fault that is capable of producing large magnitude events and that has high rate of seismic activity (Class B seismic source according to Table: 16-U of UBC-97).

Table 1. Time history record for different types of Earthquake

EARTHQUAKE	MAGNITUDE	RECORD/COMPONENT	PGA
EL-CENTRO 1979/10/15	M (6.5)	IMPAVAL/H-AEP 045 Closest to fault rupture- 16 km	0.327 g
KOBE 1995/01/16	M (6.9)	KOBE/KAK 000 Closest to fault rupture-26.4 km	0.251 g
NORTHRIDGE 1994/01/17	M (6.7)	NORTHR/ORR 360 Closest to fault rupture- 22.6 km	0.514 g

The recording stations are just near to an active fault, it is likely to be subjected to the near-fault effects. The UBC-97 takes these effects into account by defining the near source factor N_V based on the closest distance to the known seismic source. The near source factor N_V is obtained from Table: 16-T of UBC-97 as 1. Based on the seismic zone factor and soil profile type for soft soil, stiff soil and hard rock, the seismic coefficient $C_{VD}=C_V$ is obtained from Table :16-R of the UBC-97 as $C_{VD}=C_V= 0.96\ N_V$ (Soft soil), 0.64 N_V (Stiff soil) and 0.32 N_V (Hard rock).

The Fig. has shown the nature of time history with its acceleration (g) and time (t).

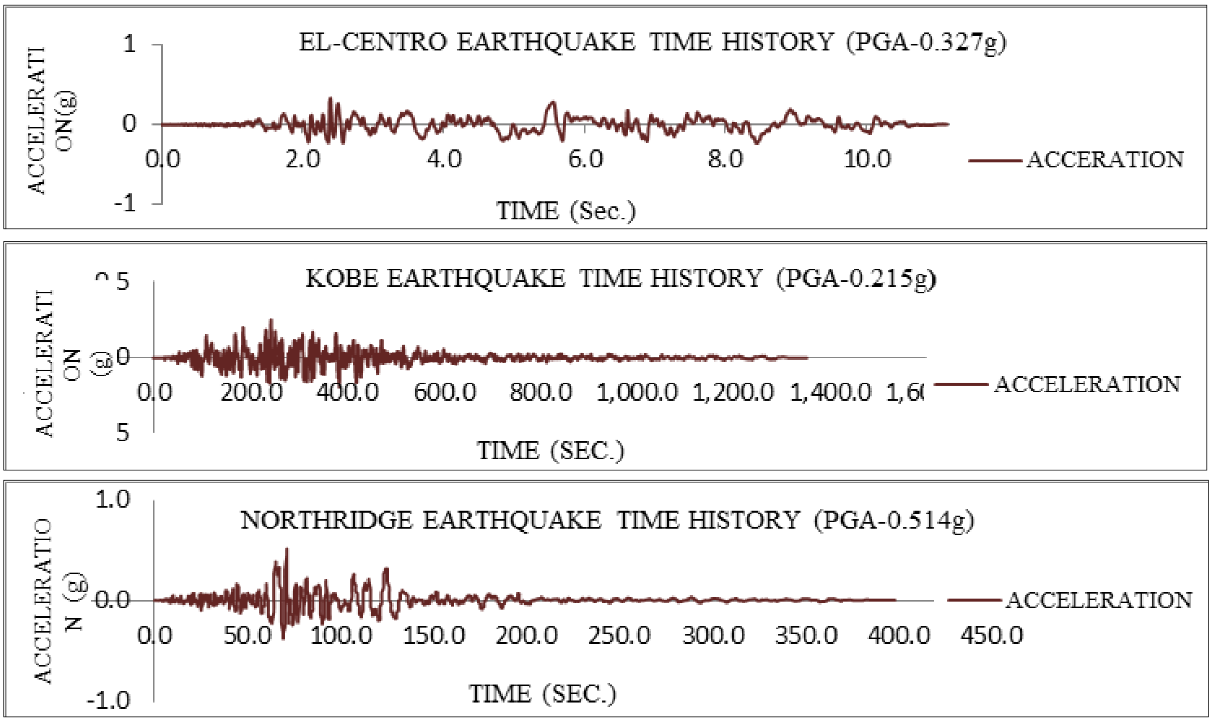
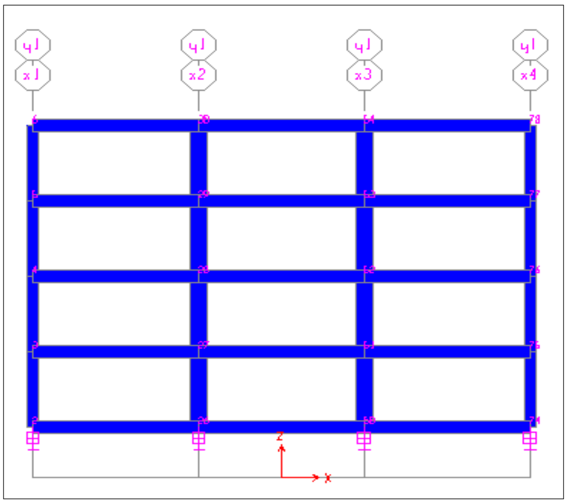


Fig. 2. Actual time history record for El-Centro, Kobe and Northridge Earthquakes.

MATHEMATICAL MODELING OF BASE-ISOLATED BUILDING STRUCTURE

For the present study, the force deformation behavior of isolator is modeled as non-linear hysteretic presented by the bi-linear model. A comparison is made for the response of fixed base and Base isolated structure also the effect of increase in time period with different soil profile. A four story RCC fixed base and base isolated (Elastomeric rubber bearing) building model prepared with design software SAP 2000 (Fig. 3). In analysis the isolator are attached at the plinth level of the structure.



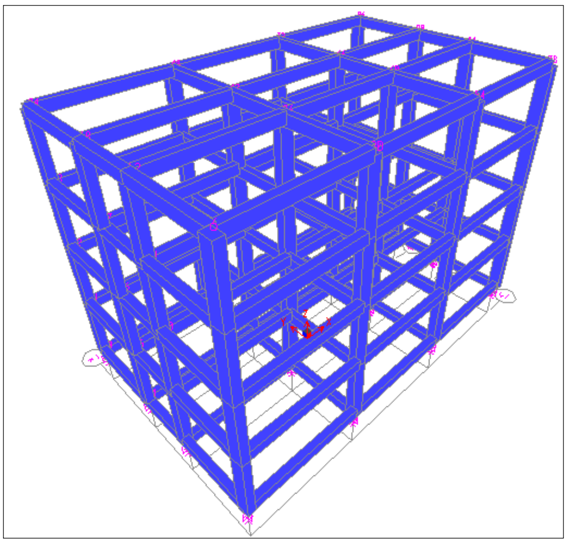


Fig. 3. Front View and 3-D view of BI building structure model

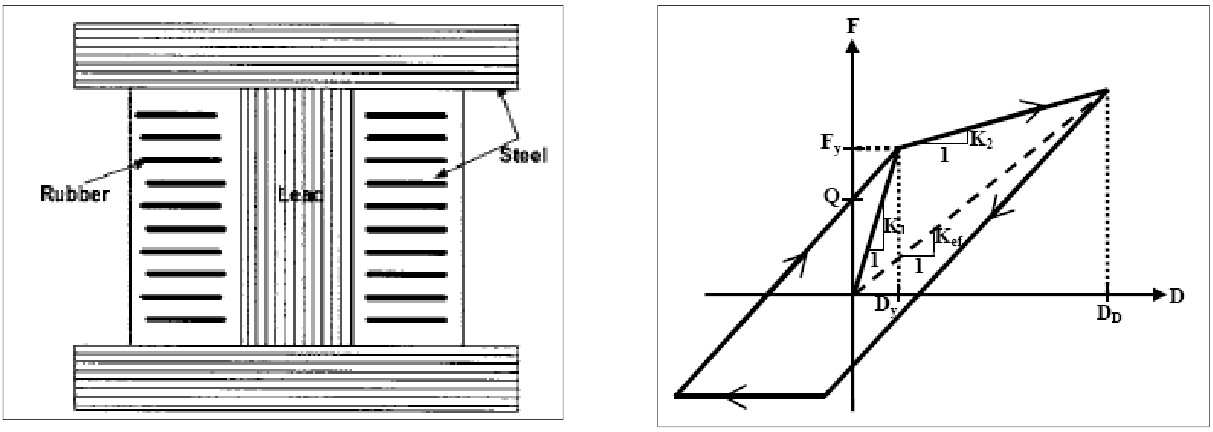


Fig. 4. Force-Deformation behavior of lead rubber bearing

a) Displacement Criteria as per UBC-97

High damping rubber bearings are composed of rubber layers and thin steel sheets. The high damping rubber bearings are composed of rubber layers and thin steel sheets. The damping is increased by adding oils, resins, or other fillers and a damping around 10%~15% can be obtained. The stiffness of the bearing is high in case of small displacements and low in case of high displacements. In this project work has follows the standard design procedure for high damping rubber bearing at MCE level effective isolation period T_M at different increasing values selected are (T_M=2, 2.5, 3, 3.3, 3.5 sec.) with effective damping β_D=0.20 has taken for study. The effective horizontal stiffness of isolation bearing is given by the equation:

$$K_{Mmin} = \frac{W}{\left(\frac{T_M}{2\pi}\right)^2 9.81} = \frac{1000}{\left(\frac{2.5}{2\pi}\right)^2 .9.81} = 643.89\ kN/m$$

Where, W is total weight carried by isolation bearing and T_M is effective isolation period assumed for MCE Level. Providing an effective isolation period

$$T_M = 2\pi\sqrt{\frac{W}{K_{eff}\cdot g}} = 2\pi\sqrt{\frac{1000}{643.89x9.81}} = 2.506\ Sec.$$

This is nearly equal to the target period. Here g is gravitational force and taken as 9.81 m/Scc^2. The damping coefficient corresponding to β_D=0.20 is B_D=1.5 according to Table A-16-C of the UBC-97.

The design displacement of an isolation system along each main horizontal axis at maximum capable earthquake (MCE) level for soft soil at El-Centro earthquake is calculated according UBC-97

$$D_D = \frac{\left(\frac{g}{4\pi^2}\right) C_{VD} T_D}{B_D} = \frac{\left(\frac{9.81}{4\pi^2}\right) 0.96x3.5}{1.5} = 0.557\ m$$

$$D_M = \frac{\left(\frac{g}{4\pi^2}\right) C_{VM} T_M}{B_D} = \frac{\left(\frac{9.81}{4\pi^2}\right) 1.2x2.5}{1.5} = 0.497\ m$$

Minimum design displacement permitted for dynamic analysis

$$D'_M = \frac{D_M}{\sqrt{1+\left(\frac{T}{T_M}\right)^2}} = \frac{0.497}{\sqrt{1+\left(\frac{0.74}{2.506}\right)^2}} = 0.476\ m$$

Where 'T' is the fixed base time period of building structure. Finally the total design displacement including additional displacement due to accidental torsion is calculated according to UBC-97 as follows:

$$D_{TD} = D_D\left(1 + y\frac{12e}{b^2+d^2}\right) = 0.557\left(1 + 4\frac{12x0.75}{8^2+15^2}\right) = 0.626\ m$$

$$D_{TM} = D_M\left(1 + y\frac{12e}{b^2+d^2}\right) = 0.497\left(1 + 4\frac{12x0.75}{8^2+15^2}\right) = 0.559\ m$$

Where b=8 m is the shortest plan dimension of the structure measured perpendicular to the longest plan dimension of the structure, which is d=15 m. Here y is the distance between the center of rigidity of the isolation system and isolation bearing placed at the sides of the plan, measured perpendicular to the direction of seismic loading under consideration, thus y=b/2=4 m in this study. Finally, e is the actual eccentricity plus the accidental eccentricity which is taken as 5 percent of the maximum building dimension perpendicular to the direction of force under consideration. The total design displacement calculated above satisfies the minimum criteria; D_{TD}=0.626 m > 1.10 D_D=0.612 m.

b) Bi-linear Hysteric model of Isolator

The non linear force deformation behavior of the isolation system is modeled through the bi-linear hysteresis loop characterized by three parameters namely:

(i) Characteristic strength Q

(ii) Initial stiffness K_1

(iii) Post yield stiffness K_2,

(iii) Yield displacement D_y (Fig. 4).

The bi-linear behavior is selected because this model can be used for all isolation systems used in practice. The force-Displacement relationship of high damping rubber bearing shows the yield force, F_y, the design displacement D_D, the effectives stiffness, K_{eff}, and characteristic force, Q.

Post yield to pre-yield stiffness ratio ($n=K_2/K_1$) depends on the material used and considered n=0.10 for rubber isolator. The elastic stiffness K_1 is difficult to measure and is usually taken to be an empirical multiple of K_2, which can be accurately estimated from the shear modulus of the rubber and the bearing design. The Post-yield stiffness of the isolation systems, K_2 is 'generally design in such a way to provide the specific value of the isolation period, T_b expressed as:

$$T_b = 2\pi\sqrt{\frac{M}{K_2}}$$

Where, M is the total mass of the base isolated structure.

NUMERICAL STUDY

Seismic response of 4-Story RC fixed base and base-isolated building structure are investigated under various real earthquake time history ground motions for non-linear isolator characteristics. The earthquake motions are selected for the studies are 1979 El-Centro, 1995 Kobe and 1994 Northridge recorded at different stations as the details are given in (Table-1). The isolation bearing characteristics for different isolation time periods are calculated according to the derived equation for rubber isolator.

Parametric Study on Isolation systems

The isolation bearing consist of an isolator to increase the natural period of the structure away from the high energy period of the earthquake, and a damper to absorb energy in order to reduce the seismic force. As the time period increases isolation parameter get changed.

In the given section parametric study have been carry out for different types of soil as per UBC-97, to study the change in values of isolation characteristics and its effect on structural behavior. As the target isolation time period changes from T=2.5 Sec. to 3.5 Sec, the mechanical characteristics values for K_1, K_2, K_{eff}, Q and F_y are found reduced in each increment in time. The values for total maximum displacement (D'_M) and total energy stored in bearing (E_{so}) increase in order T=2.5 to 3.5 Sec.

Table 2. Isolation characteristics for Soft soil with different time period of system.

Sr. No	Isolation Time Period (T) Sec.	CHARACTERISTICS OF ISOLATION BEARING								
		Tot. Max. Disp. (D'_M)	Initial Stiffness (K1)	Post Yield Stiffness (K2)	Effective Stiffness (K_{eff})	Char. Strength (Q)	Yield Disp. (D_y)	Energy Stored (E_{so})	Yield Strength (F_y)	Post Yield Stiffness Ratio (K_2/K_1)
1	2.5	0.476	4416	441.6	643.89	96.3	0.0218	72.94	107.92	0.1
2	2.7	0.517	3786	378.6	552.03	89.7	0.023	73.77	100.49	0.1
3	3	0.579	3067	306.7	447.14	74.94	0.026	74.95	91.165	0.1
4	3.3	0.64	2534	253.4	369.54	74.3	0.029	75.68	83.281	0.1
5	3.5	0.68	2253	225.3	328.51	70.2	0.0311	75.95	78.66	0.1

Table 3. Isolation characteristics for Stiff soil with different time period of system.										
		CHARACTERISTICS OF ISOLATION BEARING								
Sr. No.	Isolation Time Period (T) Sec.	Tot. Max. Disp. (D'_M)	Initial Stiffness (K_1)	Post Yield Stiffness (K_2)	Effective Stiffness (K_{eff})	Char. Strength (Q)	Yield Disp. (D_y)	Energy Stored (E_{so})	Yield Strength (F_y)	Post Yield Stiffness Ratio (K_2/K_1)
1	2.5	0.317	4416	441.6	643.89	64.1	0.0145	32.35	71.87	0.1
2	2.7	0.345	3786	378.6	552.03	59.8	0.0158	32.85	67.06	0.1
3	3	0.386	3067	306.7	447.14	54.2	0.0176	33.31	60.78	0.1
4	3.3	0.426	2534	253.4	369.54	49.5	0.0195	33.53	55.43	0.1
5	3.5	0.454	2253	225.3	328.51	46.9	0.02	33.86	52.518	0.1

Table 4. Isolation parameter of Hard rock for different time period of system										
		CHARACTERISTICS OF ISOLATION BEARING								
Sr. No.	Isolation Time Period (T) Sec.	Tot. Max. Disp. (D'_M)	Initial Stiff-ness (K1)	Post Yield Stiffness (K_2)	Effective Stiffness (Keff)	Char. Strength (Q)	Yield Disp. (D_y)	Energy Stored (E_{so})	Yield Strength (F_y)	Post Yield Stiffness Ratio (K_2/K_1)
1	2.5	0.159	4416	441.6	643.89	32.2	0.0072	8.139	35.05	0.1
2	2.7	0.172	3786	378.6	552.03	29.8	0.0078	8.165	33.43	0.1
3	3	0.193	3067	306.7	447.14	27.2	0.0088	8.327	30.38	0.1
4	3.3	0.213	2534	253.4	369.54	24.7	0.0097	8.383	27.72	0.1
5	3.5	0.227	2253	225.3	328.51	23.4	0.1039	8.46	25.25	0.1

Comparison of Fixed-Base and Base-Isolated Building Structures

In this section a comparison of earthquake response of fixed base structure with the base isolated structure is made with base-isolated building structure. The bi-linear behavior is selected in a way to represent the force-deformation behavior of the commonly used isolation system such as elastomeric bearing (i.e. lead rubber bearing).

Table 5. Output result for El-Centro, Kobe and Northridge Earthquake						
	FIXED BASE STRUCTURE			BASE ISOLATED STRUCTURE		
Earthquake	**El-Centro**	**Kobe**	**Northridge**	**El-Centro**	**Kobe**	**Northridge**
Base Shear (kN)	5202	3102	10710	2052	1713	2723
Acceleration (m^2/sec)	2.8	2.81	3.816	2.38	1.7	3.3
Displacement (m)	6.5×10^{-3}	4.2×10^{-3}	1.386×10^{-2}	0.13	0.049	0.131

The structure analyzed for above time history for soft soil condition. For the analyses structural time period has assumed 2.5 Sec. at MCE level. As result out-put it is found that the response of Base Isolated Structure is predominantly lower than Fixed Base Structure. Acceleration response at base somewhat lesser in case of isolated structure. Base displacement has increased drastically to make the structure flexible and lower damage. Represent the Base Shear response for El-Centro, Kobe and Northridge Earthquake time histories. Here earthquake response comparisons have plotted for fixed base and Base-isolated building structures. The responses are plotted for the assumed Time Period T-2.5 Sec. at the MCE level (soft soil) as per UBC-97 design criteria. The peak values for fixed and base-isolated structure are given in Table-5. During first 6 second the base shear for fixed structure gets instantly increased in El-Centro earthquake showing the undulating response but in case of base-isolated structure it shows the less and smooth response. The same behavior is obtained in Kobe earthquake during 5th to 10th second and for Northridge earthquake it happened during 10th to 15th second.

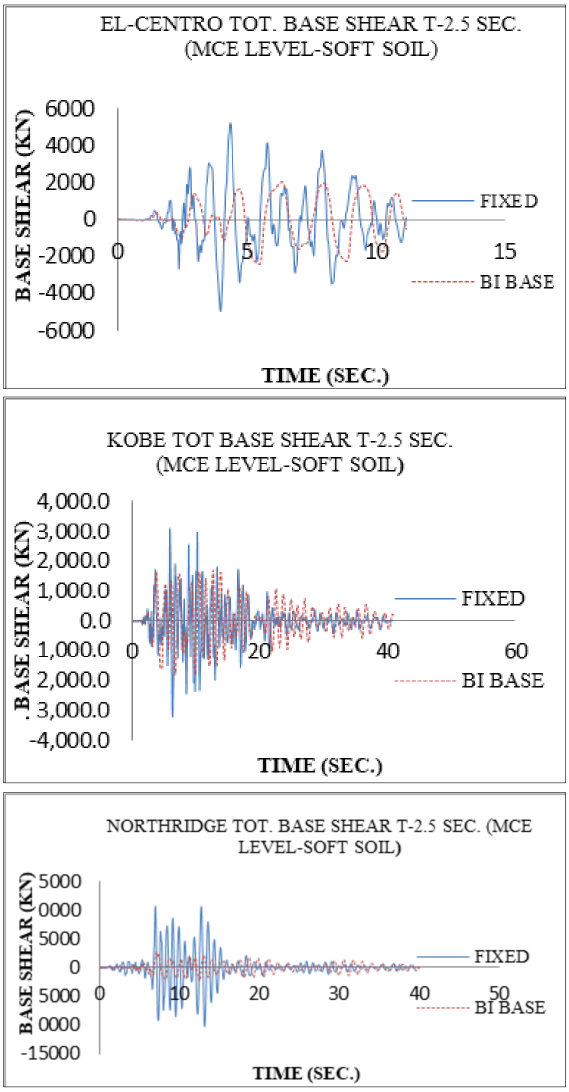
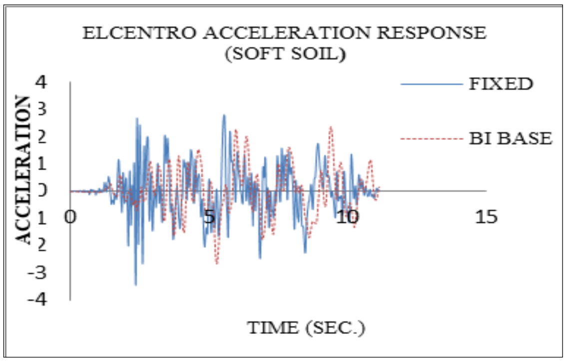
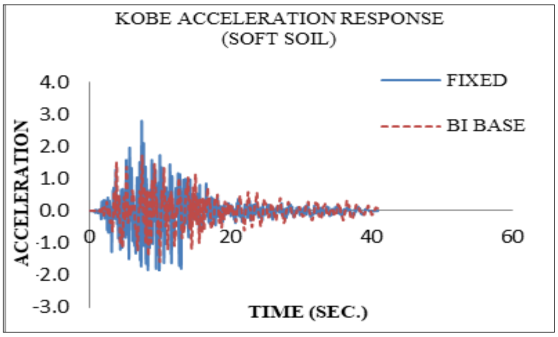


Fig. 5. Base Shear Response comparison, Fixed base and BI structure for El-Centro, Kobe and Northridge Earthquake

Fig. 6 represent the base acceleration response for fixed and base-isolated structures for El-Centro, Kobe and Northridge earthquake time history at MCE level for soft soil. The acceleration values given in Table-5. The acceleration values vary as the nature of time history has changes.





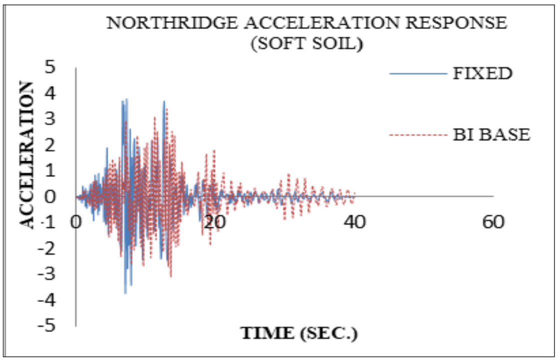


Fig. 6. Acceleration Response for Fixed base and BI base structure for El-Centro, Kobe and Northridge Earthquakes.

Fig. 7 represent the comparisons of roof top acceleration spectra for fixed base and base-isolated structures for El-Centro, Kobe and Northridge earthquake at MCE displacement level (soft soil). For base isolated structure the acceleration response get lowered suddenly in compare to fixed base structures. The response behavior for El-Centro, Kobe and Northridge earthquake has plotted the same.

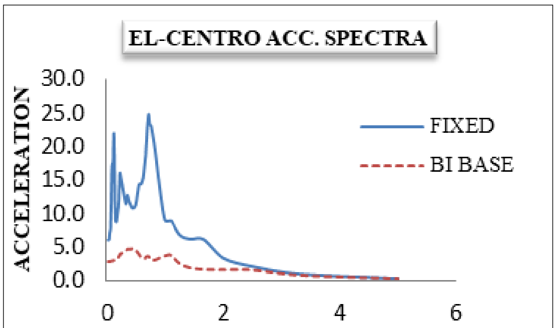


Fig. 7. (a)

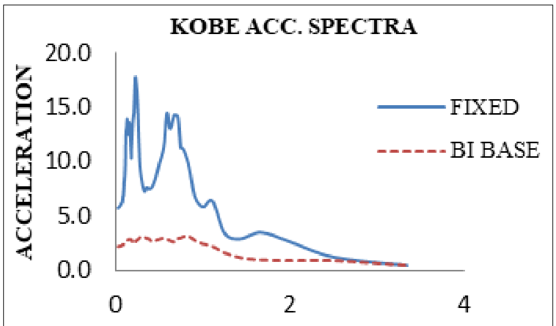


Fig. 7. (b)

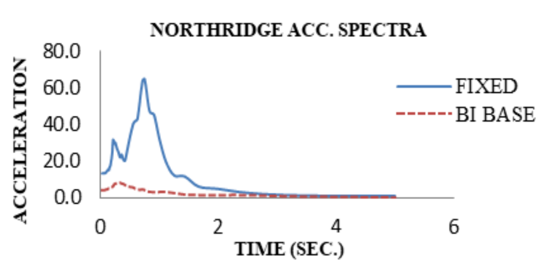
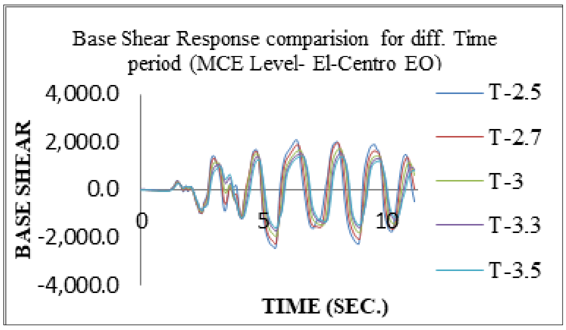


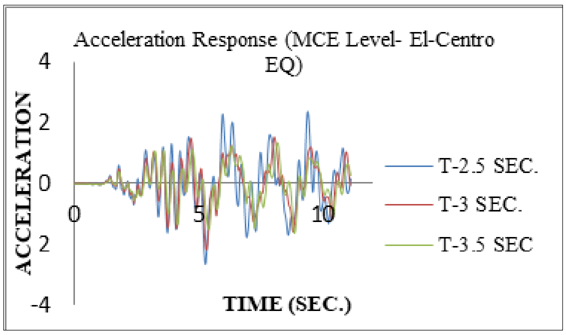
Fig. 7. (c)

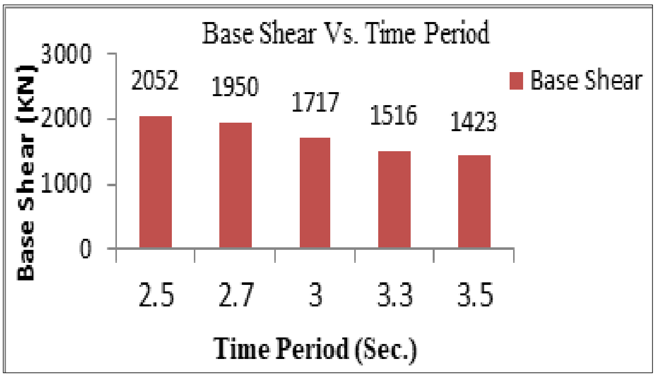
Fig. 7. Acceleration spectra for El-Centro, Kobe and Northridge Time History

Effect of Time Period of Isolation System on Response

In this project work of the isolation system the parametric study on isolation characteristics have taken to check the effect of changed target time period (MCE) on the response of structure. Time considered to calculate total displacement of the system as (T=2.5, 2.7, 3, 3.3, 3.5 Sec.). The parametric studies have been carrying out at these target time period values for different soil condition as per UBC-97. Represent the base shear response for increased time period from T-2.5 Sec. to T-3.5 Sec. From it has been found that as the time period increased the base shear response get decreased.







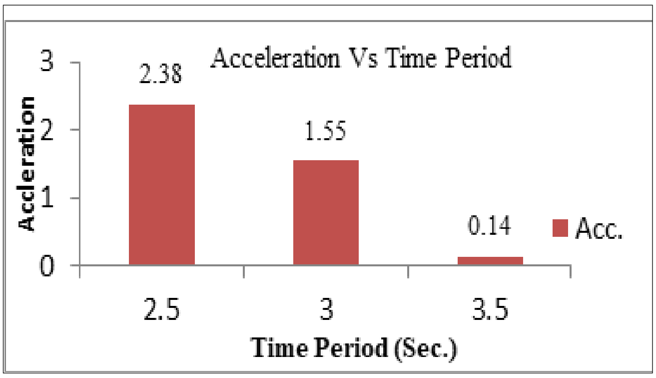


Fig. 8. Showing Base Shear Response for diff. Time period

Effect of Site Soil Condition on Structural Response

The site soil conditions for the dynamic analysis of earthquake response play a vital role. The type of soil selected from Table-16-J from UBC-97 with assuming shear wave velocity.

Table 6. Base Shear Values for El-Centro, Kobe and Northridge Earthquake for different types of soil conditions

	BASE SHEAR (KN)		
	Soft Soil	Stiff Soil	Hard Rock
El-Centro EQ.	1717	1117	821.1
Kobe EQ.	1305	942.5	555
Northridge EQ.	1811	1423	1033

As the analysis has carried out by selecting the site soil condition the result output are as shown below:

Table 7. Acceleration Values for El-Centro, Kobe and Northridge Earthquake for Different types of soil conditions.

	ACCELERATION (M/SEC^2)		
	Soft Soil	Stiff Soil	Hard Rock
El-Centro EQ.	1.356	1.11	0.834
Kobe EQ.	2.014	1.062	0.8043
Northridge EQ.	2.01	1.593	1

Representing the base shear response for El-Centro, Kobe and Northridge Earthquake. The responses plotted for UBC-97, site soil condition for soft, stiff and hard rock. From the base shear response it has found that stiff soil condition has 40% and for hard rock has nearly 50% reduction in response in compare to soft soil.

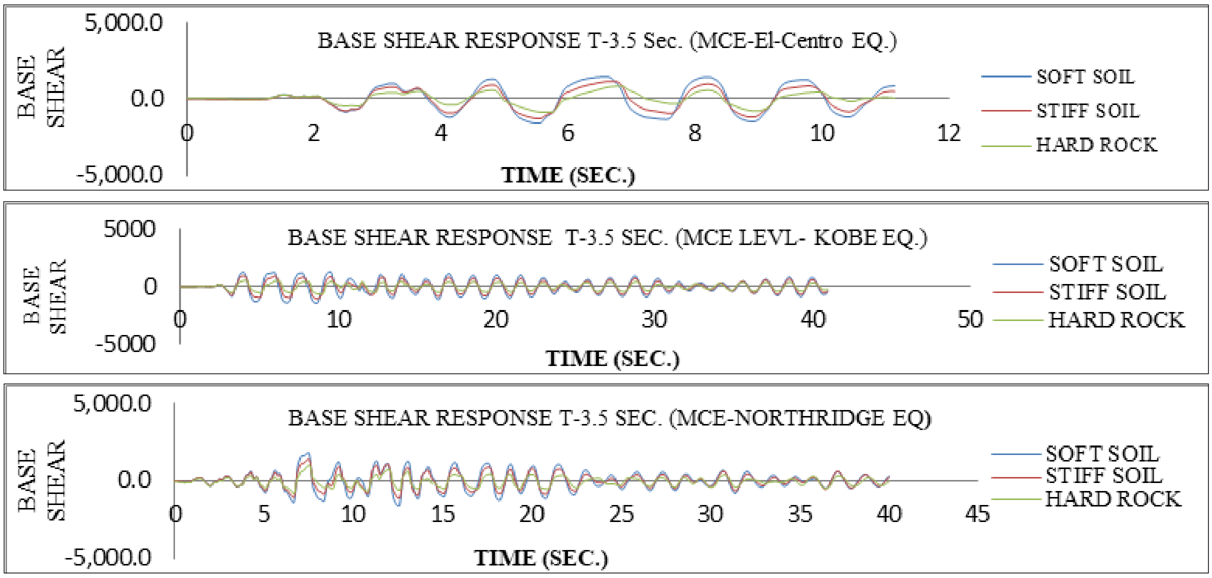


Fig. 9. Showing Base shear response for El-Centro, Kobe & Northridge EQ.

Fig.10 representing the base acceleration response for El-Centro, Kobe and Northridge Earthquake. The responses plotted for UBC-97, site soil condition for soft, stiff and hard rock. From the acceleration response it has been found that for El-Centro earthquake stiff soil has 10% and hard rock has 40% reduction in response in compare to soft soil. For Kobe earthquake stiff soil has nearly 50% and hard rock has 60% reduction in response in compare to soft soil. In case of Northridge earthquake stiff soil has nearly 40% and hard rock has 50% reduction in response in compare to soft soil.

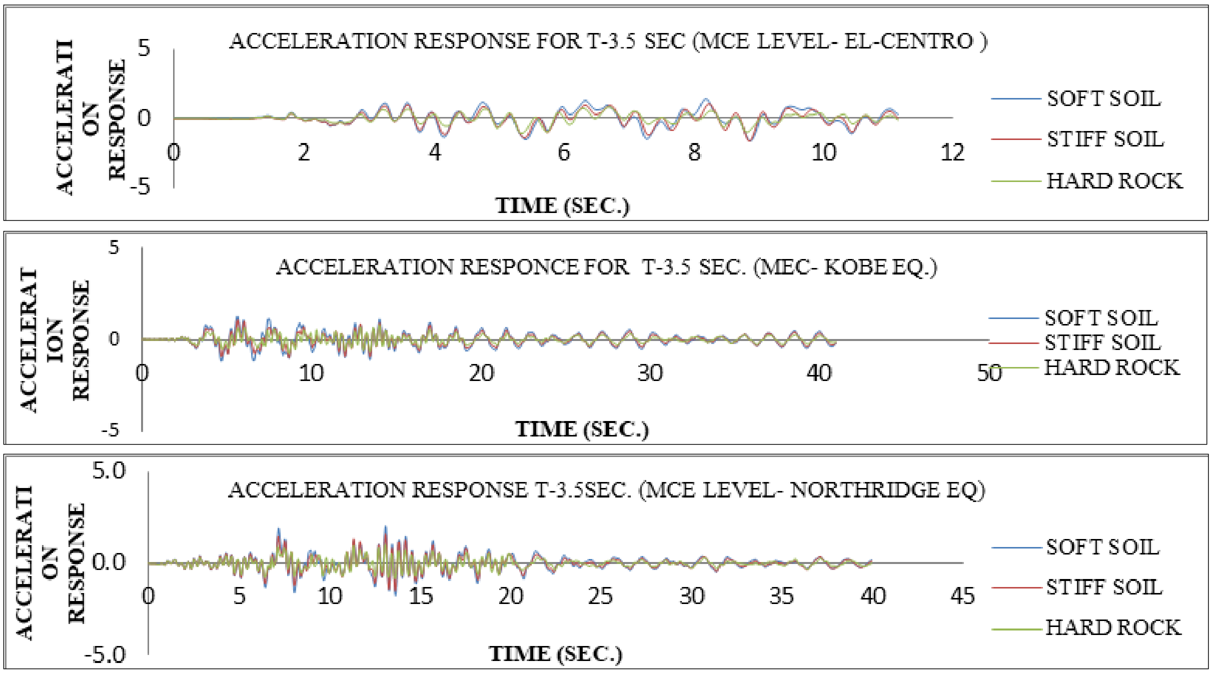


Fig. 10. Showing Base Acceleration response for El-Centro, Kobe & Northridge EQ.

Effect of Time History on Structural Response

In the given project work, model of four-story building structure isolated with rubber bearing to counteract its efficiency for different time history effect. Three-time histories of different magnitude and fault rupture distance from the site are applied through SAP-2000 base isolated building model. Different values of magnitude time histories are taken for analysis to check the effectiveness and compare its output result as per UBC-97. Representing the Base shear response for El-Centro, Kobe and Northridge earthquake (soft soil at T-3.5 Sec). From Fig. it has been found that El-Centro earthquake has 20% and for Kobe earthquake has 30% reduction in base shear in compare to Northridge earthquake.

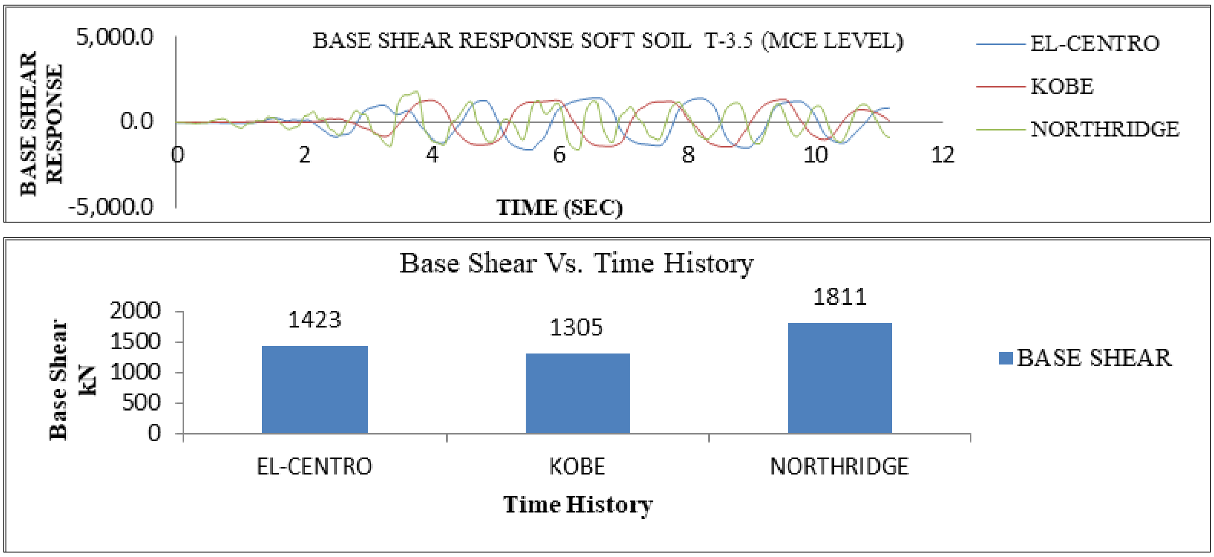


Fig. 11. Showing effect of Time History on Base shear response for Soft soil

In Fig. 12 representing the acceleration response for El-Centro, Kobe and Northridge earthquake (soft soil at T-3.5 Sec.). From Fig. it has been found that El-Centro earthquake and Kobe earthquake has nearly 35% to 40% reduction in acceleration in compare to Northridge earthquake.

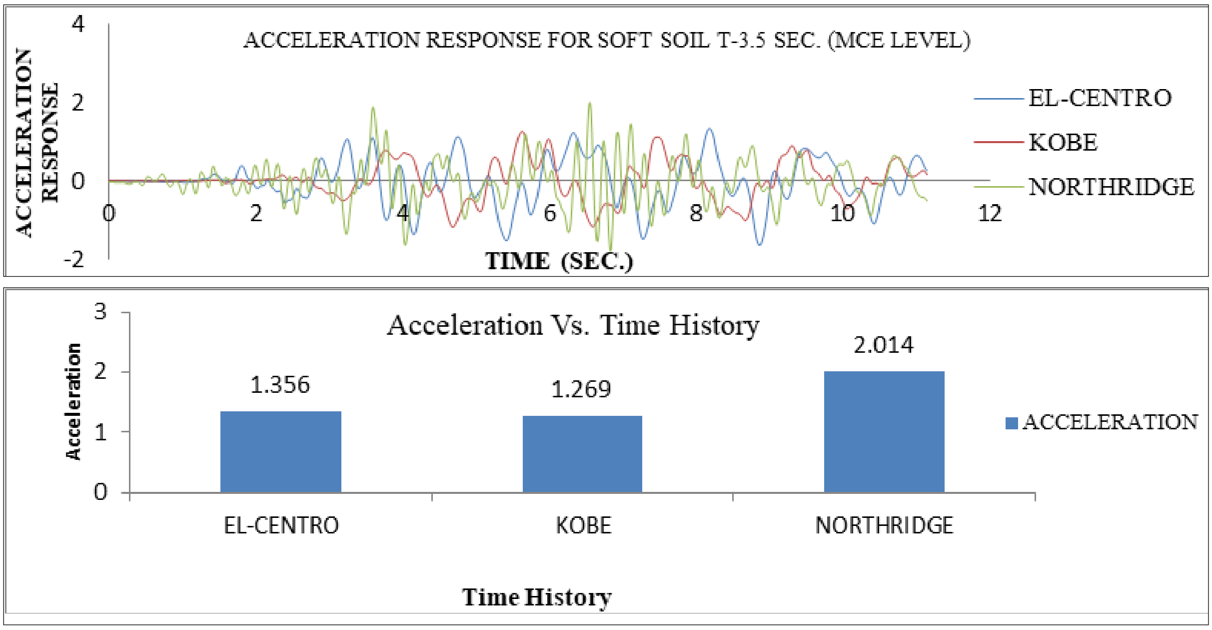


Fig. 12. Showing effect of Time History on Acceleration response for soft soil

Table 8. Showing Base shear and acceleration for soil condition as per UBC-97

	BASE SHEAR (kN)			ACCELERATION (m/Sec^2)		
Time History	El-Centro	Kobe	Northridge	El-Centro	Kobe	Northridge
Soft Soil	1423	1305	1811	1.356	1.269	2.014
Stiff Soil	1117	942.5	1423	1.11	1.062	1.36
Hard Rock	821.1	555	1033	0.834	0.804	1

Damping Effect on Isolator on Structural Response

Showing effects of increased damping on the base displacement and top top storey acceleration Due to increase in damping value of isolator it found that base displacement and storey acceleration spectra lowers down

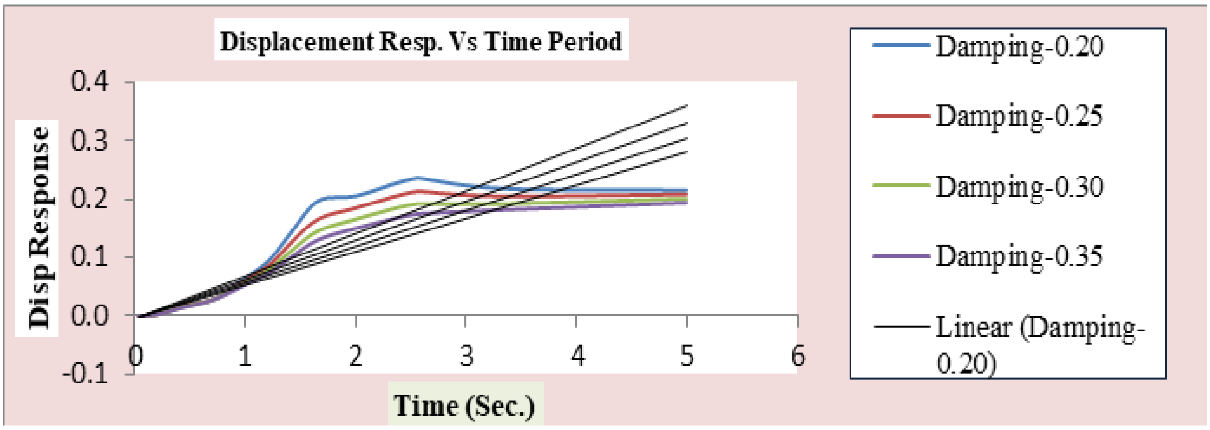


Figure 13. Showing the Base Displacement floor response spectra for different values of damping. (Soft soil, T-2.5 Sec.)

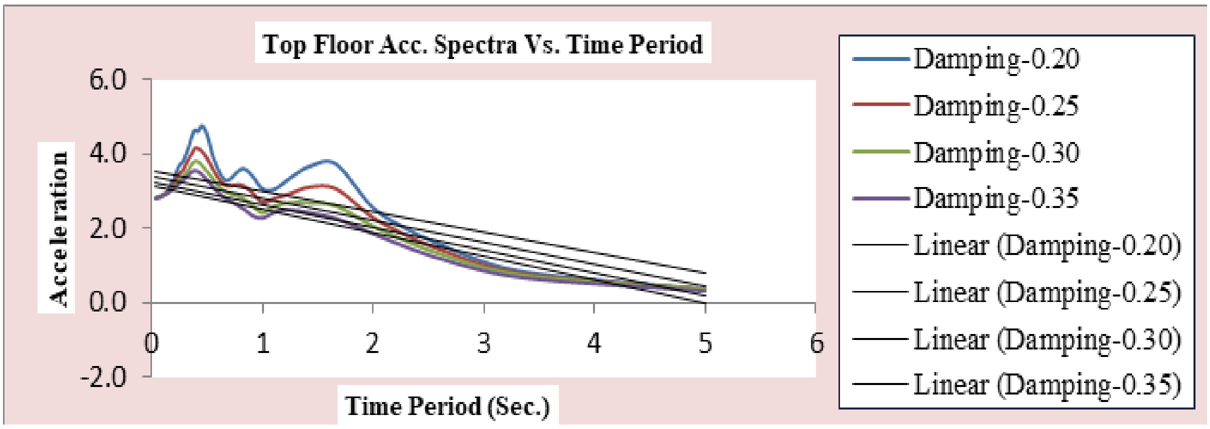


Fig. 14. Showing the Top story Acceleration floor response spectra for different values of damping (Soft soil, T-2.5 Sec.)

HYSTERESIS LOOP

The hysteresis loop associate with viscous damping is the result of dynamic hysteresis since it is related to the dynamic nature of loading. The loop area is proportional to excitation frequency. The non-linearity is well studied by hysteresis loop. In Fig. 15, shown hysteresis loop for El-Centro, Kobe and Northridge earthquake at target time period T=2.5 Sec. at MCE level for soft, stiff and hard rock soil condition as prescribed in UBC-97. The amount of energy dissipated by bearing is equal to the area covered by the hysteresis loop shown below.

Table 9.
Showing Force-Deformation values of non-linear model of bearing for Soft soil, Stiff soil and Hard rock.

	EL-CENTRO		Kobe		NORTHRIDGE	
	Force (kN)	Disp. (Cm)	Force (kN)	Disp. (Cm)	Force (kN)	Disp. (Cm)
Soft soil	154.5	12.9	115.6	4.91	147.4	11.38
Stiff soil	124.4	13.53	84.27	4.53	119.3	12.38
Hard rock	92.24	13.54	59.76	1.84	86.24	12.18

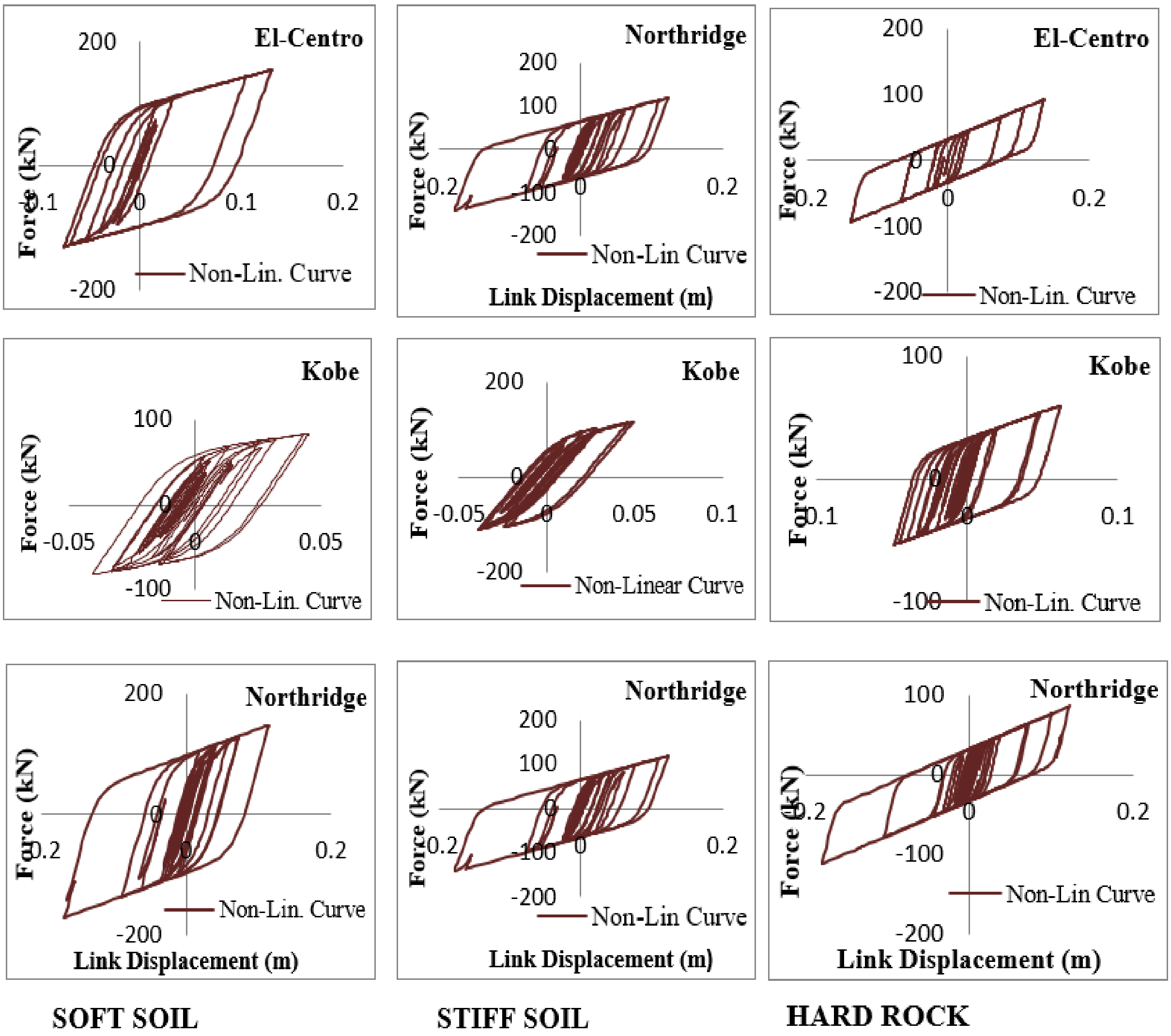


Fig. 15. Comparison of Energy dissipation of bearing for Soft soil, Stiff soil and Hard rock site condition.

CONCLUSION

The analysis of fixed base and base isolated 3-D four storey building is performed in this thesis. An exhaustive study has been performed on the performance of base isolated structures. The behavior of building structure resting on elastomeric bearing is compared with fixed base structure under maximum capable earthquake. Time history analysis has been carried out on conventional as well as Base-isolated structure to compare their base shear, acceleration and displacement response. For the analysis El-Centro, Kobe and Northridge earthquake time histories are chosen for base excitation of the structure. To study the effect of different time period of base isolator, parametric studies have been carried out for isolator for different soil condition as per UBC-97. To check the effectiveness of the isolation system, performance criteria have been carried out for fixed base isolated structure. According to analysis study, following conclusions are drawn

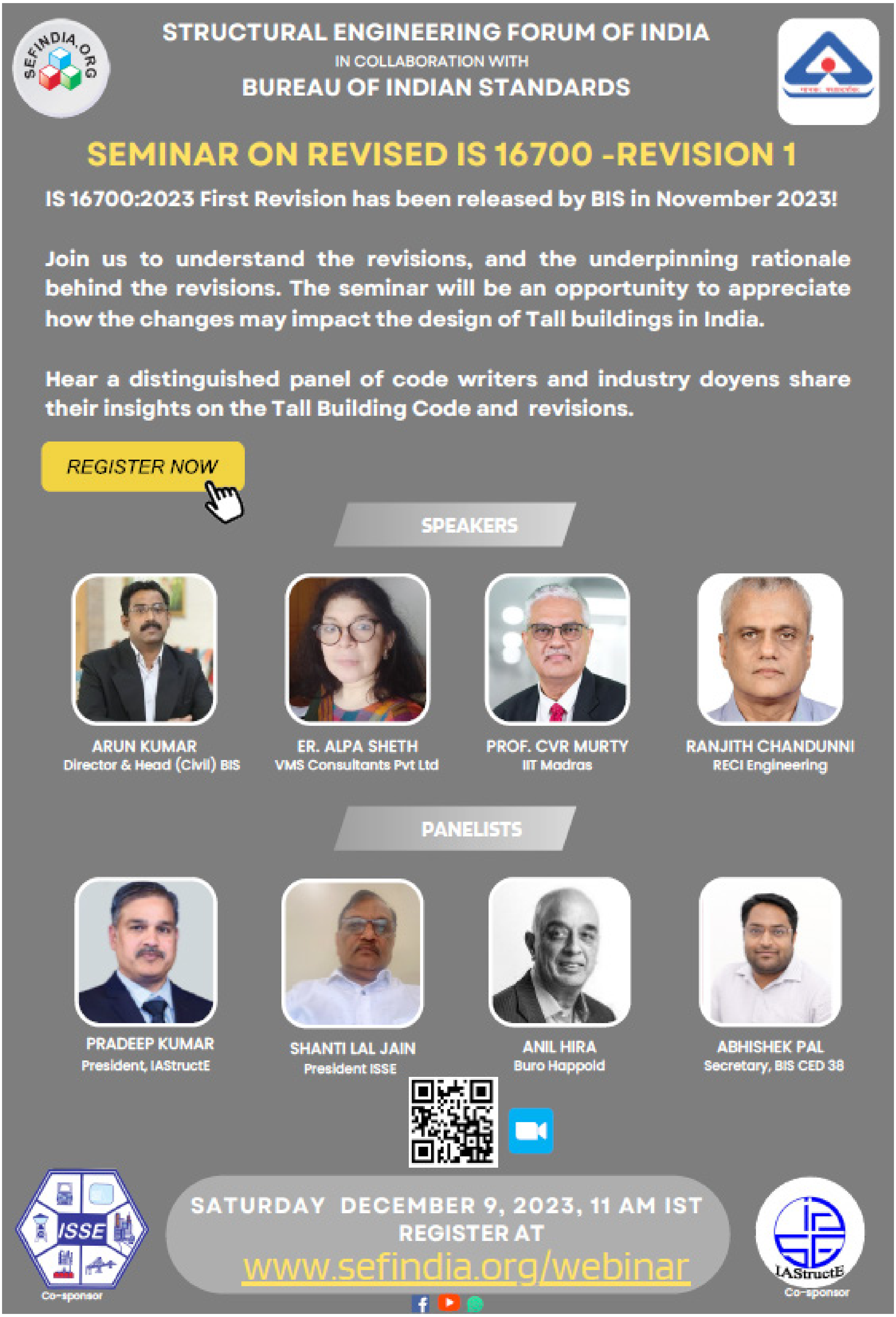
- Base isolation helps in reducing the design parameters i.e. base shear and bending moment in the structural members above the isolation interface by around 4-5 times.
- The base displacement is 2-times in soft soil strata and nearly 3-times increase in case of medium soil when compared to corresponding fixed base structure.
- Base shear and acceleration response reduces as the increase in time period and vice versa.
- During the parametric study on isolation bearing it have been found that, the total maximum displacement (D'), Yield displacement (Dy) and Energy stored in system get increased with increase in time period, also the properties like Initial

stiffness (K1), Post yield stiffness (K2), Effective stiffness (Keff), Characteristic strength (Q) get reduced with increase in time period.

- The base shear, displacement and acceleration response is higher in case of soft soil than the corresponding value for hard rock.
- Time period affects the earthquake response of the structure, as the time period increases the base shear and acceleration values found to be reducing; however the displacement increases with the same.

REFERENCE

- *ATC-17-1 (1993), "Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control." Applied Technology Council, California.*
- *Alhan C. and Gavin H. (2003), "A Parametric study of linear and non-linear passively damped seismic isolation system for Buildings", Journal of Engineering Structures, Vol. 26, Pages 485-497.*
- *Alhan C. and Gavin H. (2009), "Performance of Non-linear Base-Isolation Systems Designed According to Uniform Building Code", 5th International Advanced Technologies Symposium (IATS'09), Karabuk, Turkey.*
- *Glenn J. (2002), "Experimental Verification of Seismic Response of Building Frame with Adaptive Sliding Base-Isolation System", Journal of Structural Engineering, Vol. 128, No. 8, pp. 1037-1045.*
- *Henri G. (2003), "Fault Tolerance of Seismiactive Seismic Isolation", Journal of Structural Engineering, Vol. 129, No. 7, Pages 922-932.*
- *Henri P. (2005), "Optimal Control of Earthquake Response using Seismiactive Isolation", Journal of Engg. Mechanics, Vol. 131, No. 8, Pages 769-776.*
- *Izuru T (2005), "Stiffness-Damping Simultaneous Identification under Limited Observation", Journal of Engineering Mechanics, Vol. 131, No. 10, pp. 1027-1039.*
- *Jangid R.S. and Dutta T.K. (1993), "Performance of Base-Isolation System for Asymmetric Building Subjected to Random Excitation", Journal of Structural Engineering, Vol. 17, No. 6, Pages 443-454.*
- *Jangid R.S. and Matsagar V.A. (2004), "Influence of Isolator Characteristics on the Response of Base-Isolated Structure", Journal of Engineering Structures, Vol. 26, Pages 1735-1749.*
- *Keri L. (2006), "Estimating Seismic Demands for Isolation Bearings with Building Overturning Effects" , Journal of Structural Engineering, Vol. 132, No. 7, Pages 1118-1128.*
- *Keri L. (2009) "Problem with Rayleigh damping in base isolated building", Journal of Structural Engineering, Vol. 134, No.11, Pages 1780-1784.*
- *Keri L. and Kelly J. (2005), "Non-Linear Model For Lead-Rubber Bearing Including Axial Load Effect", Journal of Engineering Mechanics, Vol. 131, No. 12, Pages 1270-1278.*
- *Lin A. N. (1992) "Seismic performance of fixed and Base isolated steel frames", Journal of Engineering Mechanics, Vol.118, No. 5, pp. 921-941.*
- *Lee H. and Hong J. (2001), "Vertical distribution of equivalent static load for base isolated building structures", Journal of Engineering Structures, Vol. 23, Pages 1293-1306.*
- *Mark A. (2004),"Energy Balance Assessment of Base-Isolated Structures", Journal of Engineering Mechanics, Vol. 130, No.3, Pages 347-358.*
- *Naeim. F and Kelly. J. M (1999), "Design of seismic isolated structures from theory to practice", Wiley, New York.*
- *Nagarajaiah S. (1992) "Experimental study of sliding isolated structure", Journal of Structural Engineering, Vol. 118, No. 6, Pages 1666-1682.*
- *Nagrajaiah S. (1993), "Torsion In Base-Isolated Structures With Elastomeric Isolation System", Journal of Structural Engineering, Vol. 119, No. 10, Pages 2932-2951.*
- *Nagrajaiah S. (2000), "Response of Base-Isolated USC Hospital Building Northridge Earthquake", Journal of Structural Engineering, Vol. 126, No. 10, Pages 1177-1186.*
- *Nagrajaiah S. and Andrei M. (1991), "Nonlinear Dynamic Analysis of 3-D Base-Isolated Structures" Journal of Structural Engineering, Vol. 117, No. 7, Pages 2035-2054.*
- *Pradeep T. V. and D. K. Paul (2007) "Forced Deformation of Isolation Bearing", Journal of Bridge Engineering, Vol.12, No.4, Pages 527-529.*
- *Sanjay S. (2005) "Experimental study of sliding Base Isolated building with magneto rheological Damper in near fault earthquake", Journal of Structural Engineering, Vol. 131, No.7, Pages 1025-1034.*
- *Sharma A. and Jangid R. (2009), "Behavior of base isolated structures with high initial isolator Stiffness", World Academy of Science, Engineering and Technology, Paper No.50.*
- *UBC-1997, "Uniform Building Code", Vol.2, pp. 9-38 & 405-416.*



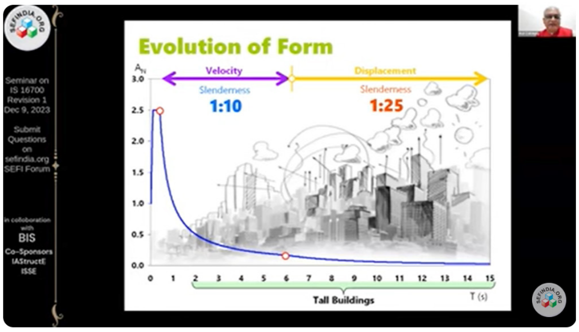
https://youtu.be/ENsMpyTg2X0?si=mKpBnoW7VS9oNede

SEMINAR ON REVISED IS 16700 -REVISION 1

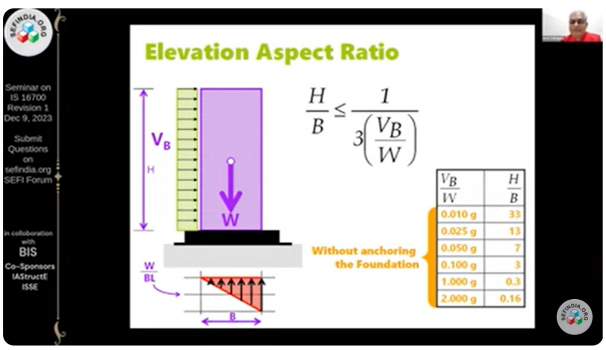
In light of the recent revision of Indian Standard IS 16700:2023 – "Criteria for Structural Safety of Tall Concrete Buildings', a detailed discussion in the form a webinar was organized by Structural Engineering Forum of India (SEFI) jointly with the Bureau of Indian Standards (BIS), in collaboration with Indian Association of Structural Engineers (IAStructE) and Indian Society of Structural Engineers (ISSE) on 9th December 2023. The session started with a warm welcome by Er. Alpa Sheth (VMS Consultants Pvt. Ltd.) to all the participants.

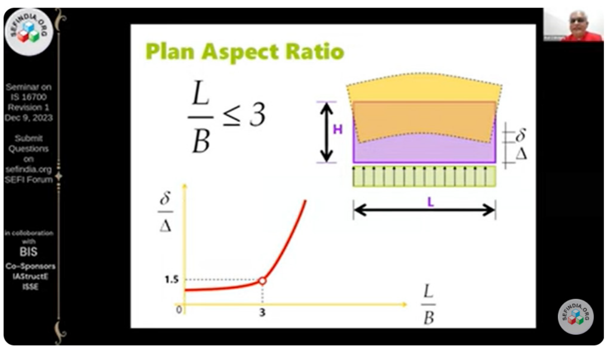
Mr. Arun Kumar (Director, and Head - Civil, BIS) started his opening remark on a very positive note highlighting the change in approach among the wider section of the construction professionals wherein they have started to appreciate the different standards and more importantly, different authorities have also started to indicate them in their relevant documents to ensure strict adherence. He mentioned that in India, about 65% of the total population is likely to shift to urban areas by the year 2050. Currently, the tall concrete structures are common in few cities, possibly due to their limited landscape. When it comes to high rise buildings, should essentially involve variety of building professionals including architects, structural designers, geotechnical specialist, MEP experts, accessibility, and sustainability experts and a strong project management team, from the concept stage of the project. At the same time, it is important to give adequate attention to address the livability aspect and maintain the quality of life of the inhabitants. He aptly mentioned that our race to going tall should also be addressing the betterment of the people.

Prof. CVR Murty (IIT Madras) started his deliberation with the fact that earlier tall buildings had their slenderness restricted to a ratio of 1:10 but over the period of time the engineering has taken new levels and the slenderness ratio of some of the high rise buildings have gone as high as 1:24. As the ratio goes higher, the relative displacement of the structure becomes somewhat more than comfortable. While there were few modifications with respect to the use of lightweight building materials or restricting the storey height, the issues of glass façade and leakage along the perimeter in adverse weather conditions, have always been matter of concern. And that is where the concept of minimum base shear comes into consideration.

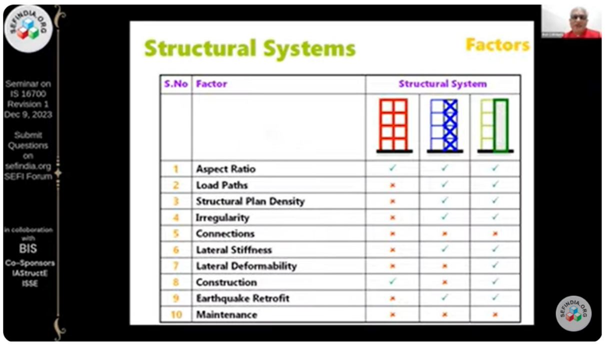


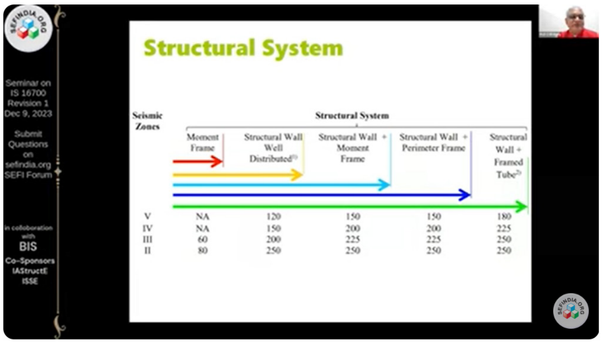
Adding on to this, Prof. Murty mentioned that if the tall buildings are not properly anchored, then one would be restricted to small size of the structure or in other words a limited value of the acceleration. Similarly, if the plan size if too large e.g., beyond a L/B ratio of 3, the criticality will be little more than comfortable. In tall buildings, this is being addressed wherein the plan length is increasing in some of the buildings. In the new standard, the allowable slenderness ratio has been restricted to 9 for zones IV and V while for zones II and III, the permissible value is 10. For structures in high seismic regions, they need to suitably anchored at the foundation level.





Prof. Murty also touch based upon the commonly designed structureal systems i.e. the frame system, the truss system and the wall-frame system, and concluded that the wall-frame system would always outperform the other two systems in terms of almost all the parametric and design requirements. The wall-frame system can also be a combination of wall & perimeter frame, wall & tube or simply tubes and the performance of the individual structures would also vary, leading to improved understanding of the behaviour. Corresponding to every type of structural system, there is a restriction of building height for a given seismic zone, beyond which a superior structural system has to be adopted.





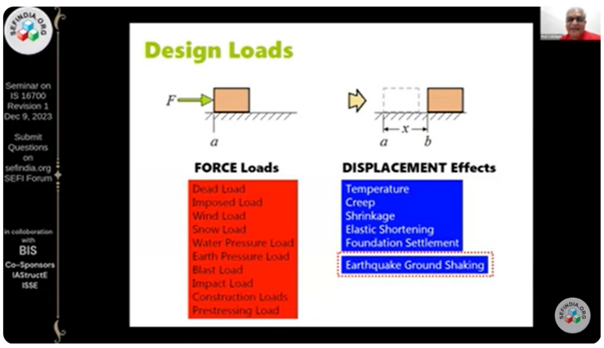
Structural plan density (SPD) in the range of 1-4% are being commonly required for normal multi storied buildings, however the same is not applicable for tall building. As a practice, it is recommended to adhere to the guidelines given in the standard for the minimum structural wall area, without creating any exception.

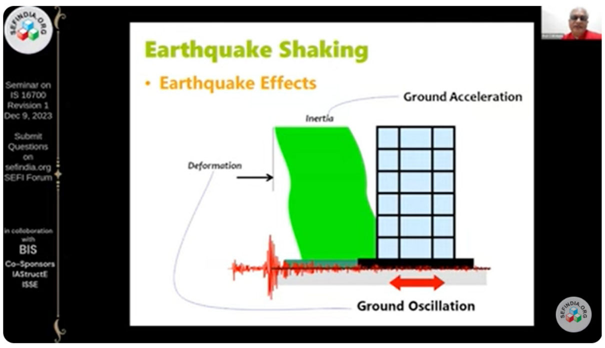
He added that in order to accommodate for livable spaces at intermediate heights of a tall structure, there would be vertical irregularities in terms of stiffness and strength, that would occur and those need to carefully addressed while designing phase of the structure itself. Another issue which has to be taken into consideration is the plan irregularity of the structure.

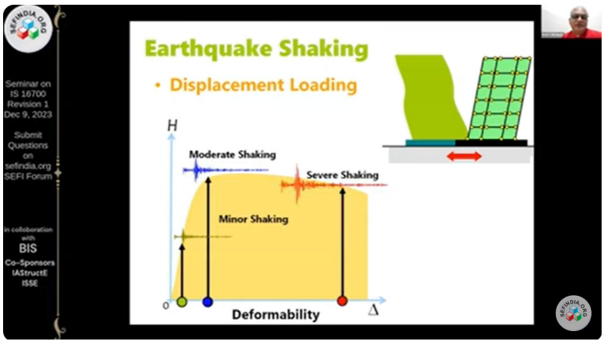
Preferably the geometry of the structure should be regular. Having said that, there ought to be certain departures from the ideal situation since member stiffness across the building would also not be uniform. He encouraged to undertake some manual exercises to make primary evaluation. Where it in not possible to completely remove the plan irregularity, it is to be ensured that the mass per unit area and the stiffness per unit area of the building across the entire plan is same.

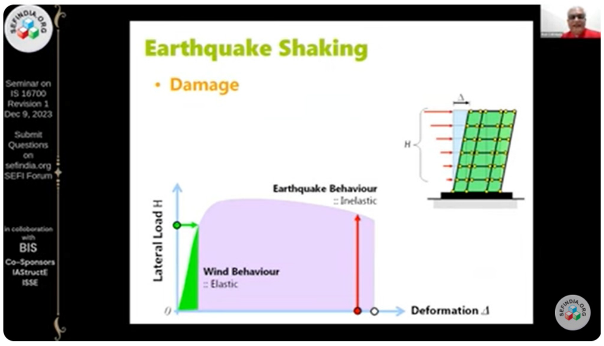
Currently the structures are being designed against earthquake considering configuration, stiffness and strength, but it is time to also plug in the concepts of ductility and deformability in the design workflow, which will be important to ensure no collapse of the structure. The future is defining the collapse mechanism and understanding energy dissipation of the structure.

Of the major loads acting on a structure, some are force loads while others are displacement effects. Adequate attention shall be given to the latter, especially for tall buildings. Earthquake shaking will be a point of concern for tall buildings because the level of shaking will indicate the deformation demand imposed on the structure. This is quite differerent from how the wind loads are to be accounted for. There is a common practice to design for one zone higher in case of earthquake, however, this may not always be a safe design to perform because the deformability of the structure is the key and if this is not satisfied, the design may be futile in the eventuality.

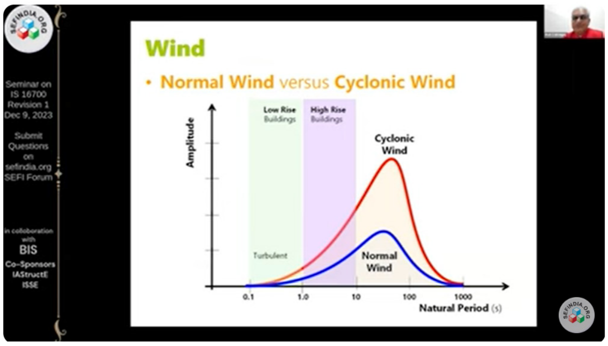


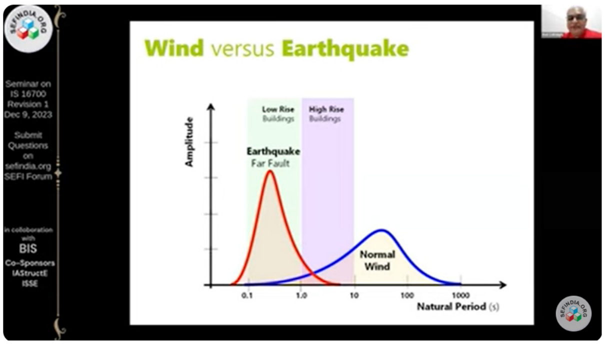


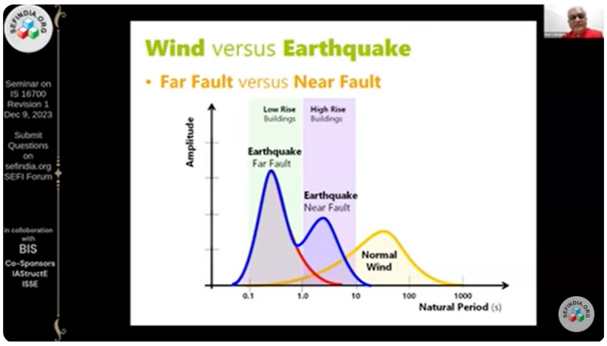




When it comes to analysis under wind load, during cyclones the frequency and the amplitude will change as compared to normal wind. However, when it comes to comparison with earthquake, the amplifications are much different for far fault earthquakes in case of low building heights. As we make the buildings taller, the effect of wind becomes more pronounced. However, in the near fault region, this logic does not hold good and earthquakes play a significant role in the design process.

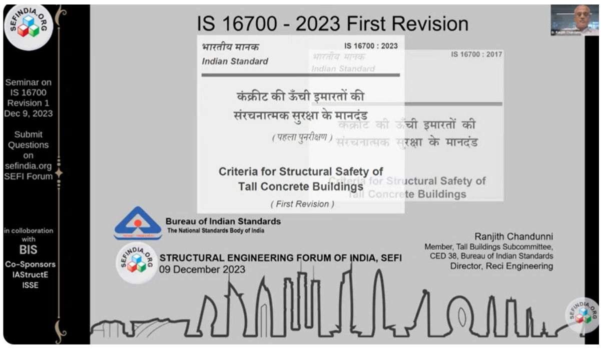




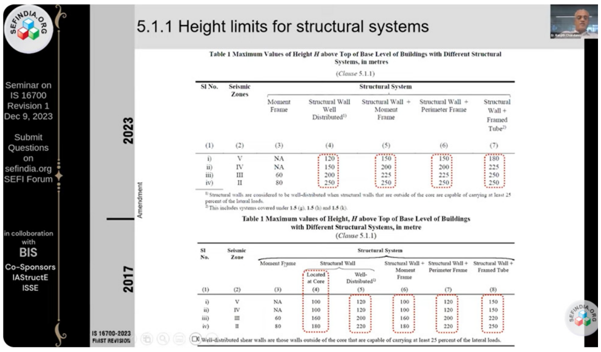


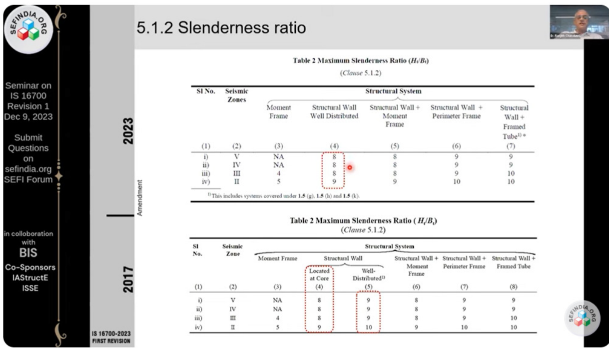
In conclusion, Prof. Murty encouraged the use of adequate size of structural members, specially building columns since large size of the columns will take us to the possibility of less or no damage under earthquake shaking.

Mr. Ranjith Chandunni (Director, RECI Engineering) in his deliberation focused on the major changes that have been made during revision of IS 16700. He mentioned that the intent of the code has remained the same – setting prescriptive parameters for satisfactory design of tall buildings with certain exceptions for the designers, provided there are checks and balances in place. Major changes which have been considered in the revision of this standard is with respect to the modification to the structural systems, wind load return period for serviceability, some changes to the vertical and lateral floor acceleration, there is a new expression for the estimation of time period, P-Delta load combinations have been specified, modification to the expression for interstorey drift stability coefficient, some changes with respect to the thickness and reinforcement for structural walls in high seismic zones and minor changes to the approval process.



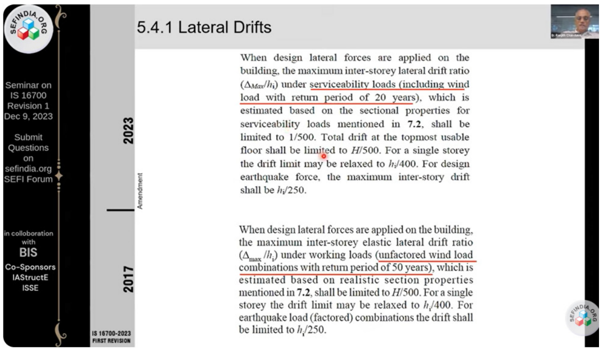
Regarding the structural systems, the earlier standard had structural walls with well distributed systems and walls located in the core area only. In the revised standard, the latter system has been dropped due to the fact that the redundancy offered by the core only system is less than the well distributed system and in case something goes wrong with the core only system, there is no alternative system to address the shortcoming. There is also a revision to height restriction of the buildings for all seismic zones. Except for moment resisting frames, the allowable building heights have been increased to 250 meter in Zone II and as the zones go higher, the building height reduces. Changes have also been made in the slenderness limits.

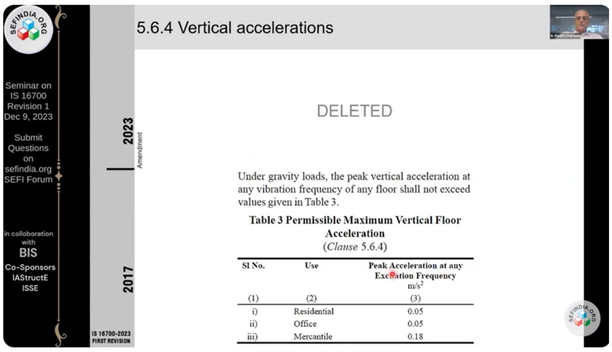




With regard to the lateral displacement due to wind, the return period has been modified to 20 years against the earlier value of 50 years. This lower level is followed for serviceability requirement, whereas for strength requirement, the earlier recommendation holds good.

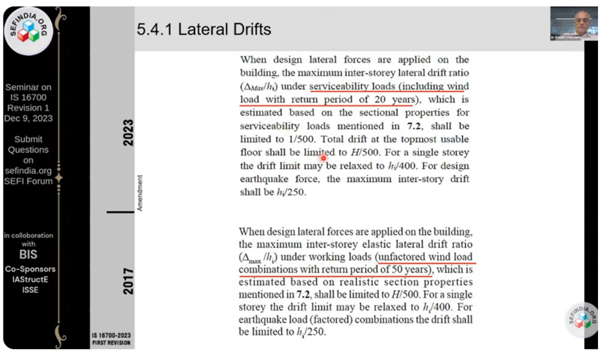
With regard to the floor acceleration, there was a requirement for peak floor accelerations, setting the limit for various types of usage. In the new standard, this requirement has been taken out completely. The requirement for floor frequency is still there as 3 Hertz.

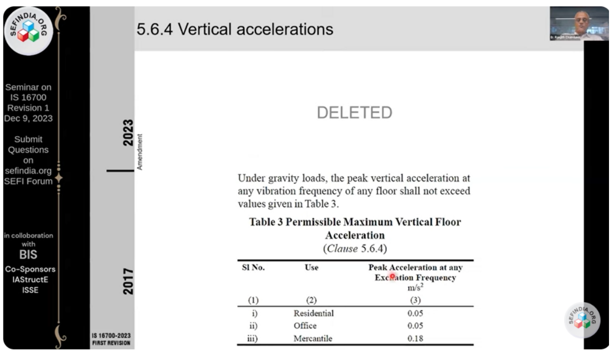




With regard to reinforcement, there was a requirement of the ratio of actual strength to design strength. This requirement has entirely been shifted to IS 13920. Also, earlier, the diameter of bar for use of couplers have been increased to 20 mm from 16 mm as in the earlier version.

Similar to vertical acceleration, there was a requirement of horizontal acceleration for any floor under wind load. The permissible value of allowable horizontal acceleration has been revised to 0.18 m/s2 for residential against the earlier value of 0.15 m/s^2. For commercial buildings, the limit has been set as 0.25 m/s^2.



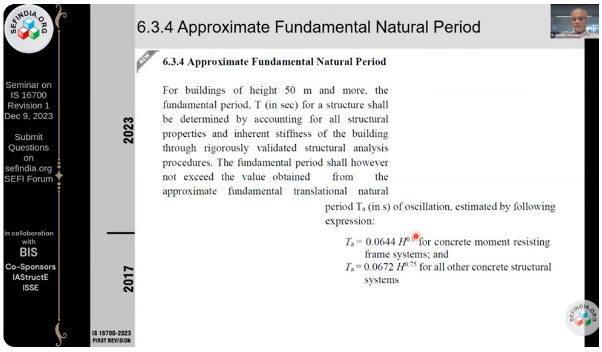


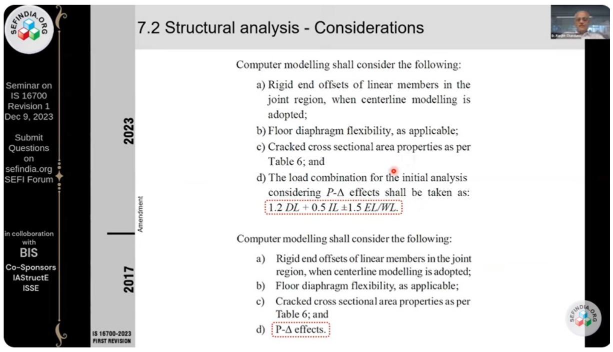
For buildings in sesmic zone V, the deterministic site specific design spectra has been made optional while the same has been withdrawn for zone IV.

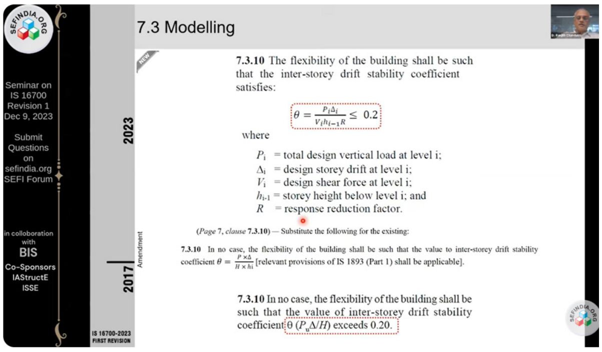
A new addition to the standard is the fundamental natural period for moment resisting frames and other systems. For structural analysis, the P-Δ effects had been made mandatory for analysis and design of tall structures in the earlier version of the standard. In the revision, the initial load combination has been specified.

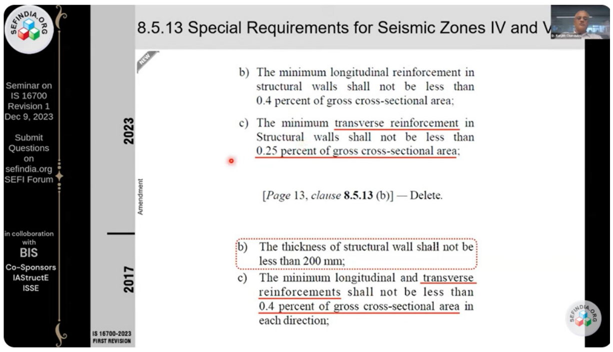
There is limit to the flexibility of the building in the form of inter storey drift stability coefficient, which has been elaborated with respect to the earlier version. With reference to the seismic detailing in high seismic zones, the earlier version had the requirement of minimum wall thickness of 200 mm and both longitudinal and transverse reinforcements as 0.4 percent of gross corss cross-sectional area. In the revision, the requirement of minimum thickness has been removed and the transverse reinforcement requirement has been relaxed to be 0.25 percent, the longitudinal reinforcement requirement remaining unchanged.

The requirement of flat slab structural wall systems have appropriately referred to the provisions of IS 1893 while for all issues related to geotechnical aspect, reference has been made to IS 1892.









Additional clauses have been provided in the annexure to address the approval process for tall buildings not meeting the requirements of the standard.

Er. Alpa Sheth started her deliberation with a very important perspective that only a handful of countries across the globe have dedicated standards for tall building design while for other countries, these are generally integrated into other standards. She highlighted that the standard is targeted more towards the practicing engineers who can appreciate and implement the standard more effectively than fresh graduate engineers.

She added that any provision of this standard which is deviated from IS 1893 or IS 13920 shall be appreciated and adherence shall be made to IS 16700 for tall buildings.

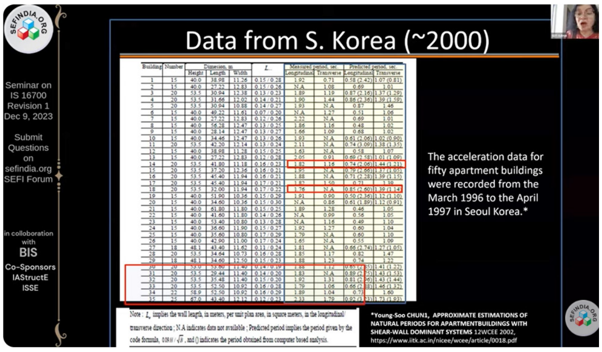
As per Er. Sheth, more than half of the tall buildings in India do not follow all the prescriptive requirements of the standard and there is no system for approval of these buildings. This is because the approval system requires to onboard building authorities having jurisdiction and presently, there is no such framework in place.

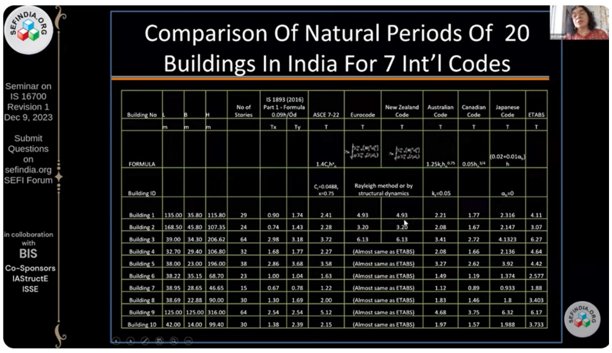
While she touch-based upon the major changes in this revision of IS 16700, the focus for the session was mainly on the genesis of the empirical equation for natural period. In the 1990s, the high rise building would be 8 to 10 storeys with thick internal partition walls of brick or concrete block masonry with conventional formwork systems, concrete walls were mostly restricted to elevators. As we transitioned into the 2000s, not just high end residentials but also the regular residential structures have gone up to as high as 20+ storeys. There has been changes in the materials used as well in the construction methodology which has witnessed a change to modular construction to address the speed, labour shortage and ensure consistency in the quality of work. Due care must be exercised to ensure that the partition walls are so designed that they do not participate in the load transfer mechanism.

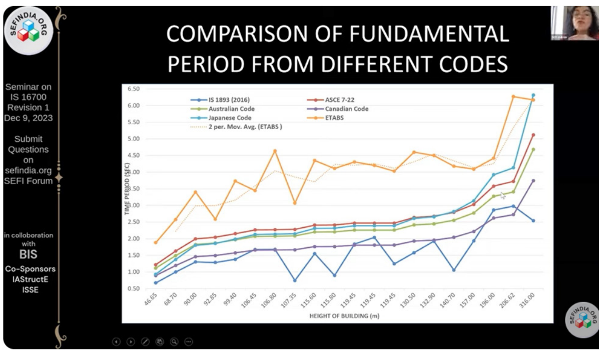
Er. Alpa Sheth mentioned that the current equation for deriving the fundamental period of a structure as per IS 1893 has been inspired by global standards and is essentially applicable to buildings up to 50 meters. She took reference of how the global standards have also evolved for tall buildings and here she highlighted the work carried out in Kores back in 2000 wherein the acceleration data for 50 apartment buildings were recorded. It was observed that there was a striking difference between the predicted natural period and the actual value. Similar activity was also taken up in Hong Kong few years later.

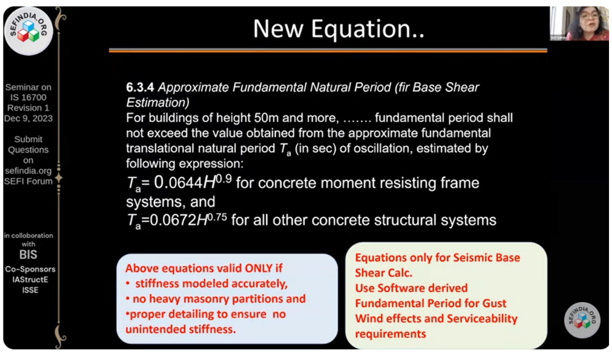
Er. Sheth shared the outcomes of her exercise wherein she had undertaken to prepare a comparative of the time period corresponding to different international standards for 20 buildings of different heights. It was observed that the ASCE gives a marginally higher fundamental period than the other standards which has also somewhat inspired the revision of IS 16700. It was also observed that there is a clear mismatch with what is actually modelled. While we calculate the time period with the empirical, it should be noted that the same is valid if the stiffness is accurately modelled, there is no heavy masonry partitions and there is no unintended stiffness.

Alternatively, it can be said that the structure should be designed at least for the value of the base shear as per the formula which will possibly give a lower time period than what the software analysis would provide.









Mr. Anil Hira (Buro Happold) added that the IS 16700 is not a prescriptive document which gives direction as to how to design, rather it is a document which forms the basis for being on the right track for compliance. With further inputs and experience, it can be further expanded and also modified as the need be. He encouraged that the first step is to get the concepts clarified, instead of focusing too much on the analytical model. Our effort should also be consistent to minimize the carbon footprint on the environment and build efficient buildings.

Dr. R Pradeep Kumar (President IAStructE) in his deliberation mentioned that the standard is very streamlined in terms of its recommendation for building height and slenderness ratio based on seismic zone and also the type of building structural system to be adopted. However, it is still little conservative in terms of the recommendation for natural period. And this would lead to more robust structures and as a result leading to higher carbon footprint. His recommendation was to test the buildings which are being constructed in India, gather the data and come up with more accurate natural period. This is no longer a challenge since we are well equipped, and the technology knowhow is available. He added that the clause on code exceeding building within the code could be misleading in few cases and requires adequate explanation. Also, more clarity would be required for the expert review panel and the criteria thereof.

Mr. Shanti Lal Jain (President ISSE) added that standards are getting developed but the implementation on ground is still a challenge. His recommendation was to BIS to create awareness among the practicing engineers. Another perspective was to also introduce a guideline for architects and approval bodies, so that the correct knowhow is available with the right stakeholders.

This was followed by a very detailed panel discussion and the questions were duly addressed by the esteemed speakers and panelists.

STRENGTHENING URBAN RESILIENCE: INSIGHTS FROM NIDM AND DDMA'S TRAINING PROGRAM

In a concerted effort to fortify urban areas against the growing threat of disasters, the National Institute of Disaster Management (NIDM), in collaboration with the Delhi Disaster Management Authority (DDMA), organized a three-day training program on "Developing Disaster Risk Resilience in Cities for Urban Local Bodies." This comprehensive initiative, held from December 27 to 29, 2022, aimed to equip senior officials from diverse government departments with the knowledge and strategies essential for enhancing disaster resilience in urban landscapes.

The three-day training program by NIDM and DDMA proved to be a pivotal step towards creating disaster-resilient urban areas. The collaborative efforts of senior officials from various government departments, coupled with expert presentations, exemplify a commitment to building safer and more resilient cities in the face of evolving environmental challenges. The insights gained from this program will undoubtedly contribute to a more prepared and resilient urban landscape.

SEISMIC DESIGN AND ASSESSMENT OF POST INSTALLED REBAR CONNECTIONS FOR RETROFITTING OF STRUCTURES

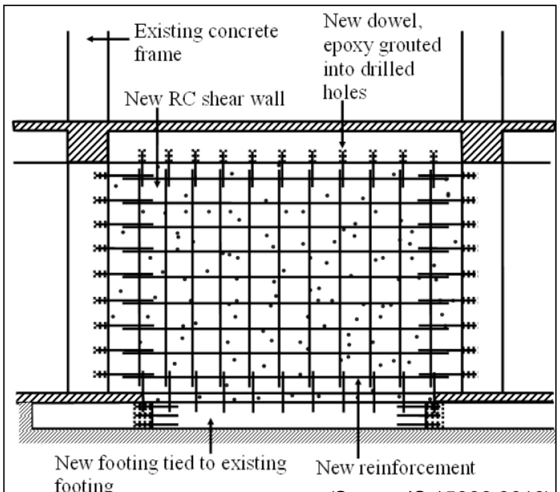
Dr. Giovacchino Genesio
Hilti Entwicklungsgesellschaft
Kaufering, Germany

Shounak Mitra
Hilti India Pvt. Ltd.

INTRODUCTION

Earthquake, for sure, is one of nature's most unpredictable hazard. Over the past so many years, we have witnessed a staggering rise in seismic activity all over the globe. In India, we are sitting on a seismically active zone, with more than 59% of the land being susceptible to earthquakes (as per the Vulnerability Atlas of India [1]). In India and its neighboring countries (within 300 km), there have been more than 2700 incidents of earthquakes of magnitude 4 and above which have recorded over the last 10 years, making it an average of 22 earthquakes per month. While standards are being upgraded continuously to take into consideration higher seismic performance of structures, there is also a need to strengthen many existing structures to meet the current seismic demand. Also, it may be required to address the deficiency of multiple structures arising out of several reasons like ageing, change of usage, increase in the load, construction errors, etc.

The seismic retrofitting of structures can be done either by increasing the seismic capacity (e.g., stiffening existing structures, strengthening the members, enhancing ductility, reducing irregularity) and/or by improving the seismic ductility of the structure (i.e., strengthening vs. brittle failure mechanisms). Other advances techniques aim the reduction of the seismic demand on the building (i.e., isolating the structure or introducing damping elements). Strengthening techniques may include interventions at a global level (e.g., addition of shear walls or bracing, thickening of walls, base isolation, etc.) or at member level (strengthening of deficient members like jacketing of columns or beams, strengthening of foundations, etc.) (Fig. 1 and Fig. 2).



(Source IS 15988:2013)

Fig. 1: Retrofitting of Structure using Shear Wall

When existing reinforced concrete member need to be connected to new elements or additional concrete is needed to increase sections of existing members the use of post installed reinforcing bars (rebars) becomes an integral part of the application. The application being critical to ensure desired seismic performance of the structure, a basic requirement is also to ensure that the connection to the existing concrete member also has adequate seismic performance during earthquake events.

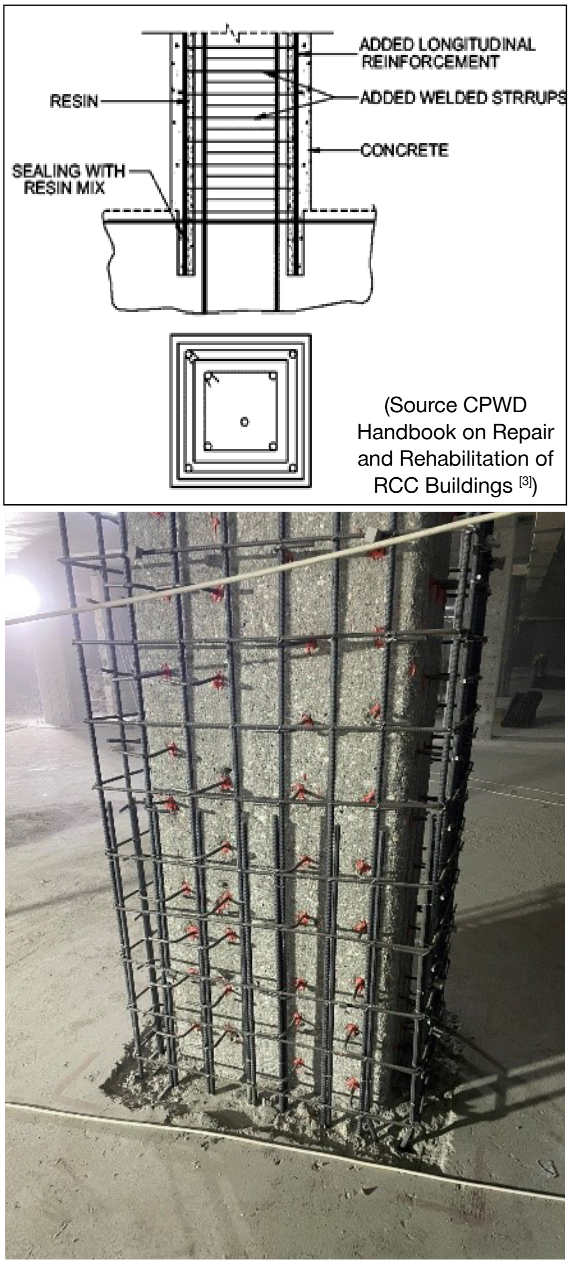


Fig. 2: Strengthening of structures using concrete jacketing

Post-installed rebar connections involve installation of deformed reinforcing bars in holes drilled in concrete filled with injectable mortars. The reinforcing bars are embedded in adhesives in holes drilled into existing concrete member and are cast in new concrete on the other side (Fig. 3a). In concrete-to-concrete connections using post-installed technology, the bars are typically embedded as required to develop the tension yield strength of the reinforcing steel. The fundamental principle of any post installed concrete to concrete connection is that it should at least behave as a cast-in connection.

To this end the performance of the mortar used and its interaction with the reinforcing bars and the concrete is of key importance (Fig. 3b).

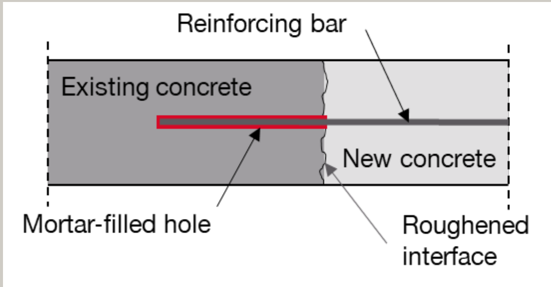


Fig. 3a - Post-installed rebar application (typ.)

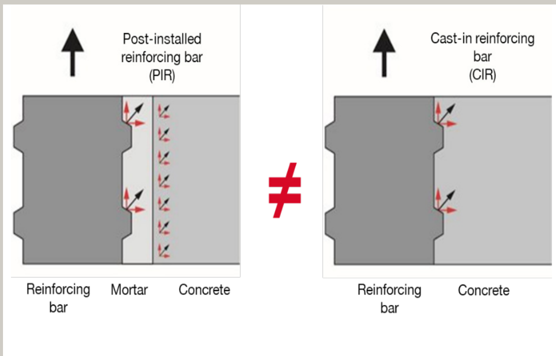


Fig. 3b - Difference between cast-in and post-installed rebars in terms of loading transfer mechanism

Fig. 3: Post-installed reinforcing bars embedded in concrete

FUNDAMENTAL OF QUALIFICATION AND DESIGN POST-INSTALLED REBARS

There are always three imperatives to ensure safety of any post installed connection – product assessment/qualification, correct design, and proper installation.

As a result of extensive research and development over the last 3 decades, we have seen a lot of progress in terms of parallel evolution of qualifications and design provisions (Fig. 4).

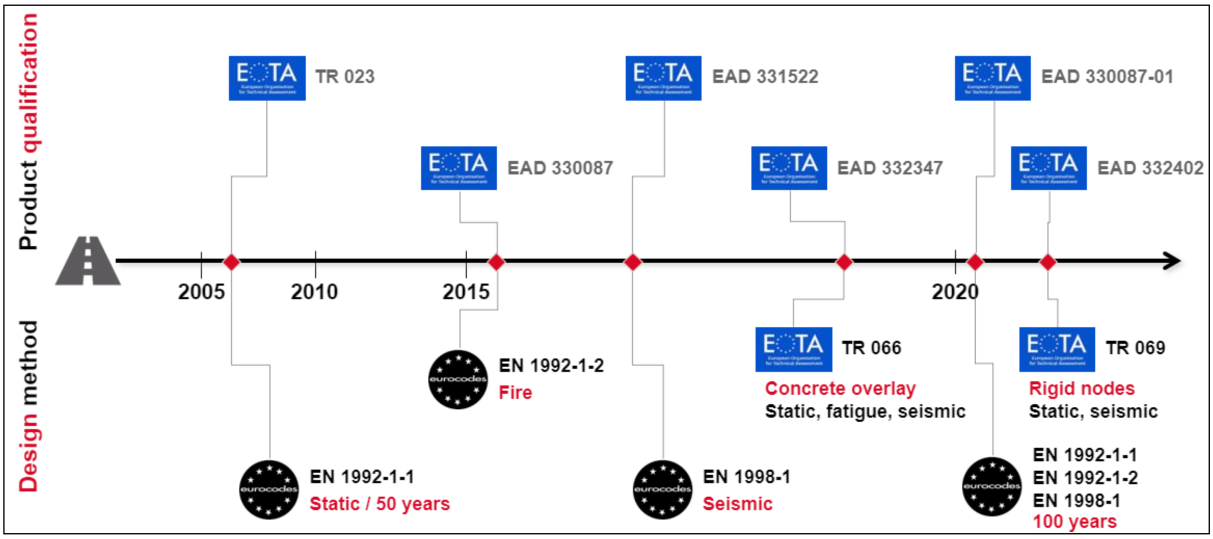


Fig. 4: Development of design standards for post installed rebar connections

A post installed rebar connection can be broadly classified as end anchorage (Fig. 5), splice connection (Fig. 6) and shear connectors or concrete overlays (Fig. 7)

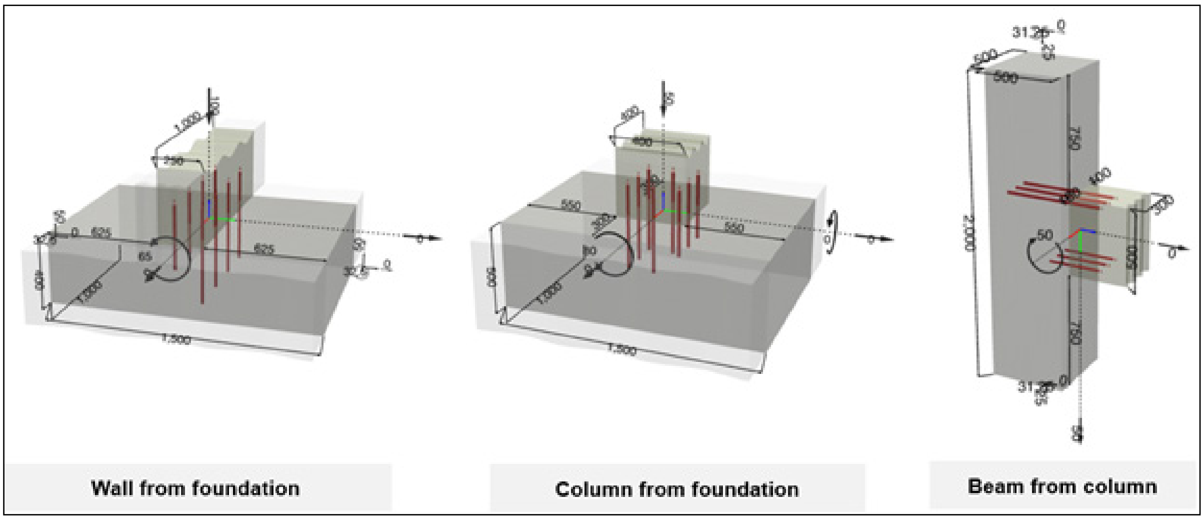


Fig. 5: Examples of end anchorage for post installed rebar connection

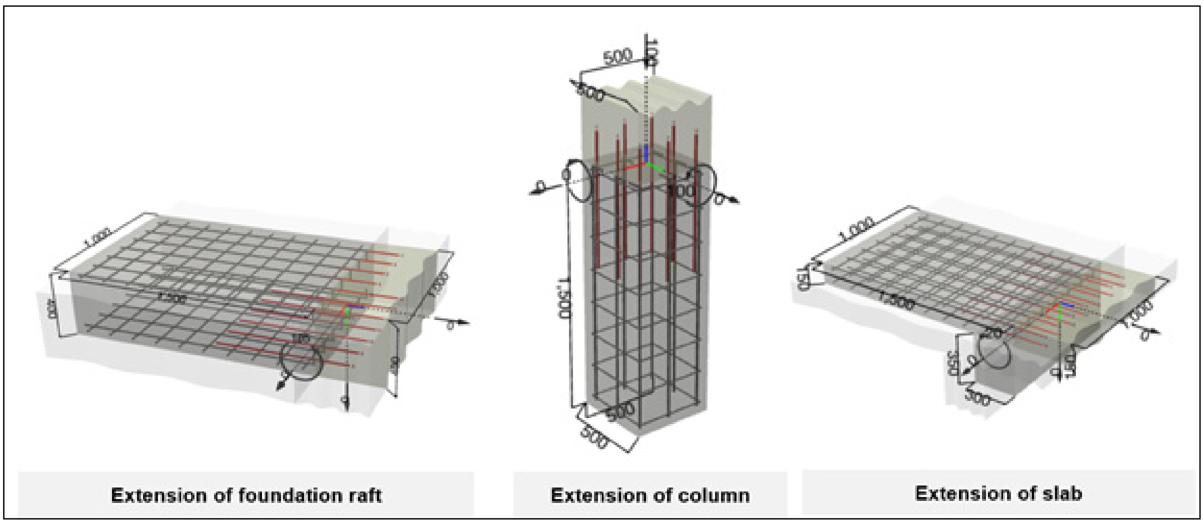


Fig. 6: Examples of splice connections for post installed rebars

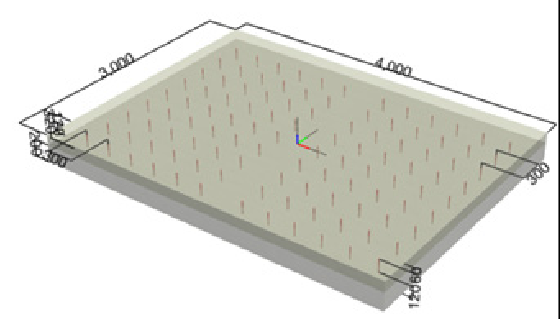


Fig. 7: Examples of shear-friction applications

REGULATORY FRAMEWORK FOR ASSESSMENT/ QUALIFICATION OF POST INSTALLED REBAR CONNECTIONS –

Assessment of post-installed rebars for equivalency to cast-in bars

To allow the use of post-installed reinforcing bar systems, verification of the compatibility of the post-installed bars with existing and neighboring cast-in bars in terms of strength, stiffness, and serviceability is required. Refer to Spieth, 2002 [4] and Genesio et al. (2017) [5] for more details and the scientific background. Furthermore, the performance of post-installed reinforcing bars is strongly linked to the mortar performance and its robustness in different installation conditions (e.g., temperature, humidity) as well as being sensitive to jobsite conditions (e.g., improper hole cleaning or/ and injection, corrosive environment), loading conditions (e.g., freeze-thaw cycles, sustained loading at high temperature, cyclic seismic loading), quality and type of equipment used for installation, and depth and diameter of the application. All these considerations point to the necessity for appropriate product qualification requirements aiming at ensuring that the behavior and performance of a post-installed reinforcement connection is similar to that of cast-in one.

Over the past three decades, extensive research work has led to the development of qualification procedures for the post installed connections, to prove their equivalence to cast-in rebars in terms of load vs. displacement behavior, bond-splitting resistance and robustness as related to installation, environmental, and loading conditions. The European Assessment Document (EAD) 330087 [6] issued by the European Organization for Technical Assessment (EOTA) provides comprehensive guideline in terms of performance assessment under static loading, fire exposure and seismic loading.

A post-installed rebar system assessed according to EAD 330087 [6] can be used following the principles of the reinforced concrete design standards EN 1992-1-1 [7] and EN 1998-1 [8] for the calculation of lap splices (Fig. 6) and anchorage lengths (Fig. 5) of longitudinal reinforcement as well as shear-friction applications, when rebars are used as dowel (Fig. 7).

Typically, the same equations are valid for both static and seismic design (refer to EN 1998-1 [8], sect. 5.6). However, for seismic design, additional requirements for reinforcement detailing are usually provided. These include increase of anchorage length to account for steel yielding and strain penetration at the onset of potential plastic hinges. Also, it takes into consideration the inclusion of seismic hooks at the end of anchorage bars to improve the confinement of the nodal zone as well as to guarantee a more stable cyclic behavior, where a sufficient straight anchorage length cannot be provided. These requirements are mainly motivated by the need to avoid a possible pullout failure.

In regions where enough confinement of the tensioned bar(s) cannot be provided, radial stresses may induce splitting cracks in the cover and/or between bars located on the same splitting plane and reduce their pullout resistance. The assessment of post-installed

> “The behavior and performance of a post-installed reinforcement connection is similar to that of cast-in one”

reinforcing bars under cyclic (seismic) loading is conducted by comparing the performance of the system in the near (splitting failure) and far (pullout) edge conditions The adhesive used must ensure that under cyclic loading the performance of the system is still equivalent to a cast-in bar.

EAD 330087[6] provides a comprehensive test protocol for seismic testing of post installed rebars which include bond strength under seismic loading and test for minimum concrete cover. Technical details and scientific background are provided by Simons (2007) [9] and Genesio et al. (2019) [10].

To evaluate the behavior under cyclic loading in far-edge conditions tests are performed in displacement controlled set-up using constant slip protocol, which consists of application of ten displacement cycles between a specific value of push and pull, followed by residual tension load test. The limiting value of displacements for the load cycles shall be 1.5 mm for diameter of rebar less than 25 mm, 2.0 mm for rebar diameter between 25 mm and 40 mm and 3.0 mm beyond 40 mm rebar diameter.

The cyclic behavior of post-installed reinforcing bars has been extensively investigated by Simons (2007) [9] using the same testing and assessment procedures developed by Eligehausen et al. (1983) [11] to investigate the bond strength of cast-in bars degradation under cyclic loading. The reference bond degradation curve (i.e., bond strength measured at the cycle n vs. bond strength at first cycle) for cast-in bars is the black dashed line shown Fig. 8b), which is valid for ten “push/pull” cycles between $\pm s_u$, where su corresponds to the displacement at peak load measured in reference monotonic pullout test with confined setup. This curve fits rather well the test results of Eligehausen et al. (1983) [11] as well (Fig. 8a).

It is worth mentioning that this loading protocol does not reflect the real seismic demand on a reinforcing bar, but a well reproducible and idealized condition under which the bond strength degradation of a post-installed bar system can be conservatively assessed and compared with the performance of cast-in bars.

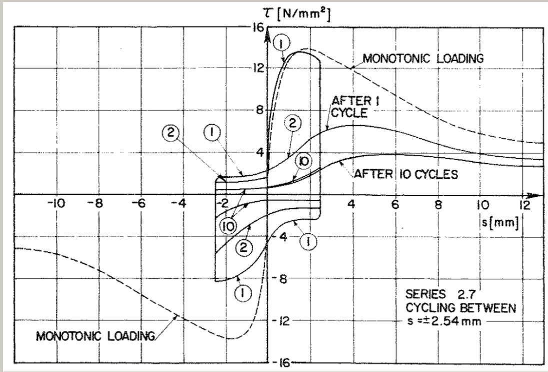


Fig. 8a - hysteretic behavior of cast-in reinforcing bars (typ), Eligehausen et al. (1983) [8]

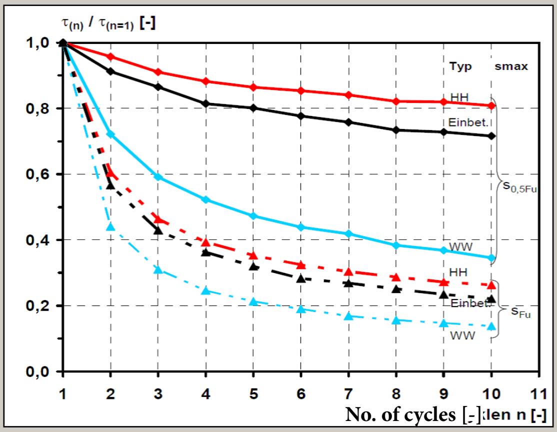


Fig. 8b - comparison of bond strength degradation of cast-in vs. post-installed reinforcing bars, Simons (2007) [9]

Fig. 8: Cyclic load protocol and assessment for post-installed reinforcing bar seismic qualification

Further, for seismic loading, cyclic tests are conducted to determine the splitting resistance of post installed rebars, as this failure mode is likely to be decisive in near edge conditions and in presence of dense reinforcement. The tests are performed using the Beam End Test set-up (BET) and unconfined set-up (Fig. 9). Details and validation of this specimen and setup are discussed by Rex et al. (2018) [12]. The test shall be performed in displacement control with increasing slip protocol (ISP) (see Fig. 8), which consists of application of three displacement cycles between 0 and maximum axial displacement (i.e., at pull-out) followed by a residual tension test. The maximum axial displacement shall be derived from monotonic tests with cast-in rebar. The assessment is based on the comparison between the cyclic performance of the post-installed reinforcing bars and the monotonic load-displacement behavior of cast-in bars as related to peak strength, dissipated energy calculated as the area below the cast-in bar monotonic curve and the envelope of the hysteretic curves obtained with post-installed reinforcing bars and residual resistance at maximum axial displacement.

Fig. 9a - Typical BET specimen

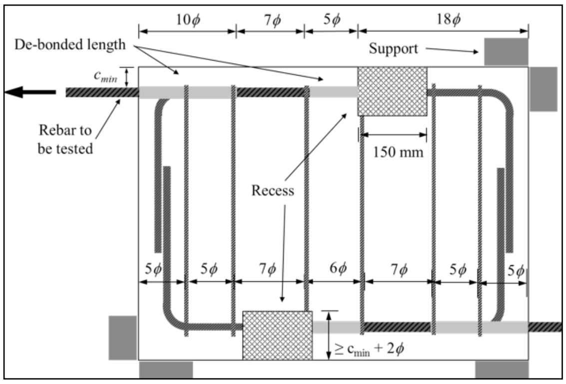


Fig. 9b - Schematic of BET specimen suitable for testing of post-installed rebars

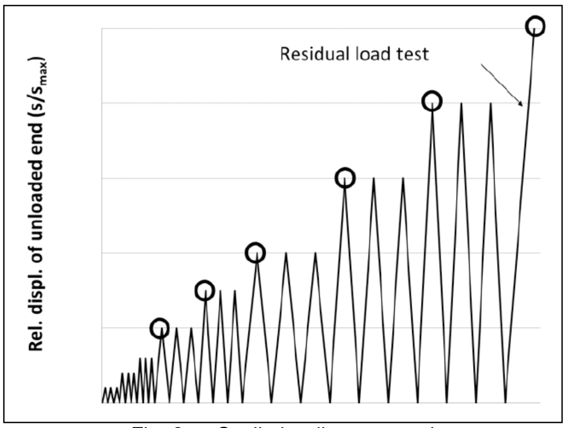


Fig. 9c - Cyclic loading protocol

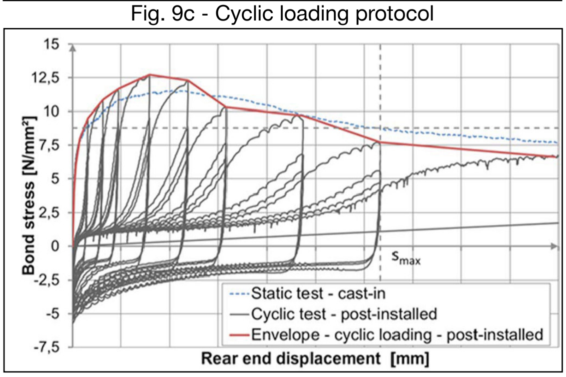


Fig. 9d -Typical cyclic response of post-installed rebars compared to cast-in bar

Fig. 9: Beam End Test (BET) Set-up (Source EAD 330087 [6])

Assessment of product specific performance of post-installed rebars

Research has shown that post installed rebars, in end anchorages, can behave better than cast-in rebars if high strength mortar is used, but this could never be leveraged owing to the design limitation where the designer was restricted to use the bond strength of cast-in bars. Experimental evidence (Rex et al., 2018 [12]) has demonstrated that the bond strength of high performing mortar system allow the increase of splitting dominant field beyond the ratio $c_d/\phi > 3$ (Fig. 10a). At the same time, it is important to highlight that the difference in bond strength between post-installed and cast-in bars decreases with increasing anchorage length (Fig. 10b) due to the shear lap effect. On this research basis, the EOTA has developed the EAD 332402 [13], [14] and [15] that establishes the rules for the assessment of enhanced bond-splitting performance of post installed systems, following the principles explained in the fib Model Code 2010 [16]. It covers both static and seismic loading, for a design working life up to 100 years.

The basic assessment mainly consists of:

1. derivation of bond-splitting equation and assessment of all relevant products parameters as function of: concrete strength f_{ck}, bar diameter Ø, minimum cover c_d, maximum cover c_{max} as defined in the fib Model Code 2010 [16] with a beam-end-tests similar to the one shown in Fig. 10
2. Assessment of the bond strength degradation with increasing anchorage length.
3. Pullout strength assessed according to the EAD 330499 [17] as upper limit of the splitting resistance (Fig. 11) including the sensitivity to cracked concrete, temperature, sustained load as well as other environmental and loading influencing factors.

The seismic assessment follows the principles explained in the previous section of this paper with the difference that the benchmark behavior is not the cast-in bar anymore, but the static performance of the post-installed rebar system under consideration (refer also to Cattaneo et al., 2023 [18]).

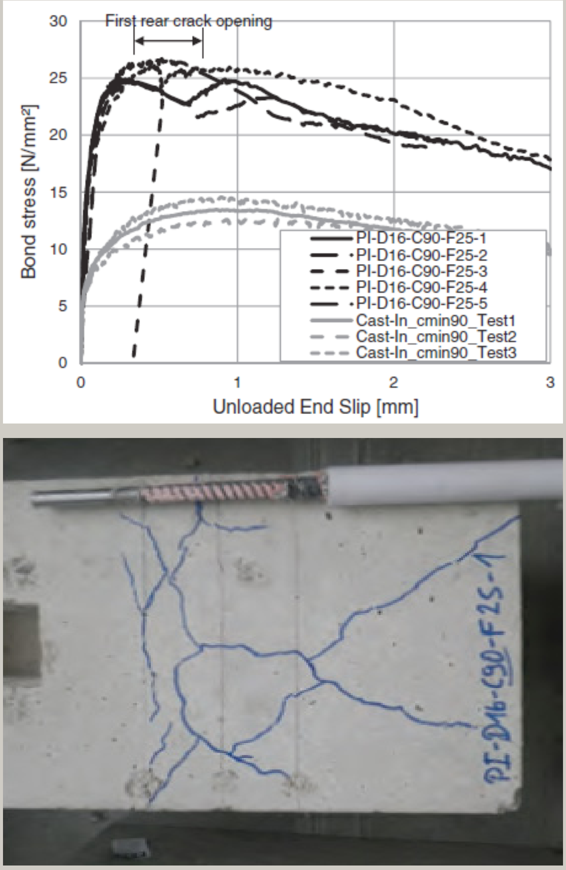


Fig. 10a - BET with small anchorage length *(7∅) and c_d/ ∅ ≈ 5.6)*

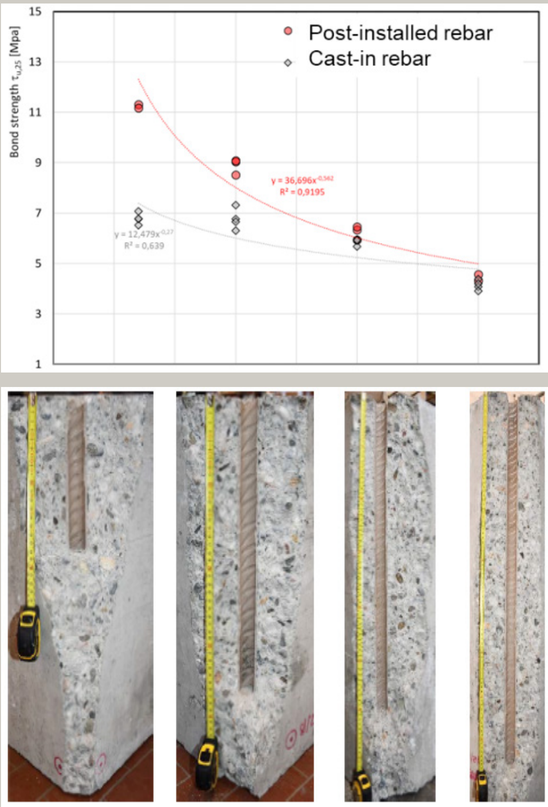


Fig. 10b - Influence of anchorage length on bond strength

Fig. 10: Experimental evidence of superior bond strength of post-installed vs. cast-in rebars

The bond-strength of a post-installed rebar system assessed according to the EAD 332402 [13], [14] and [15] is schematically shown in Fig. 12.

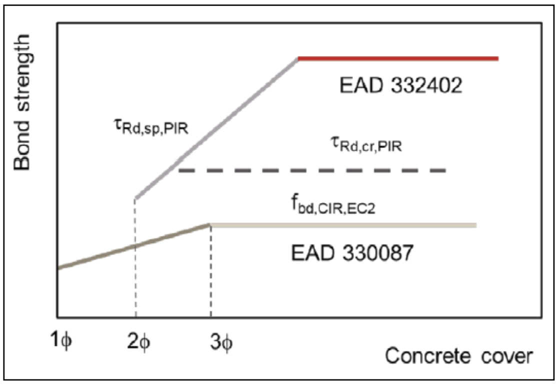


Fig. 12a - Bond strength as function of the concrete cover

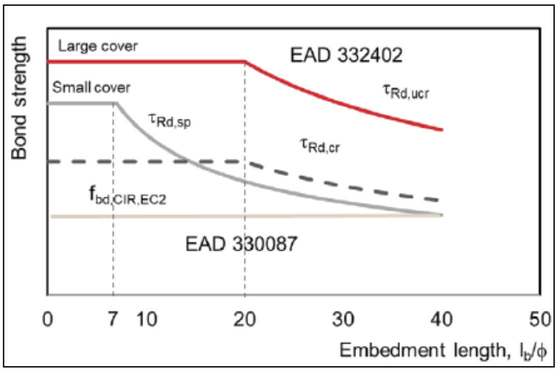


Fig. 12b -Bond strength as function of the anchorage depth

Fig. 12: Influence of confinement and anchorage depth on bond splitting resistance of post-installed rebars assessed according to the EAD 330087 and EAD 332402

Design as equal to cast-in

EAD 330087[6] covers post installed connections designed in accordance with EN 1992-1-1 [7] for design of concrete structures. The standard covers the design provision for calculation of anchorage and lap splices lengths for connections with cast-in rebars. With the basic assumption that post-installed connections should behave at least as cast-in connections, the provisions of EN 1992-1-1 [7] can be extended for design of post installed connections with a few modifications. Fundamentally, the design of anchorages and lap splices as per Eurocode, has the following formulation

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot \alpha_6 \cdot l_{b,reqd} \geq \alpha_{lb} \cdot l_{b,min}$$

Where,

- α_1 is for the effect of the form of the bars assuming adequate cover
- α_2 is for the effect of concrete minimum cover

- α_3 is for the effect of confinement by transverse reinforcement
- α_4 is for the influence of one or more welded transverse bars along the design anchorage length l_{bd}
- α_5 is for the effect of the pressure transverse to the plane of splitting along the design anchorage length
- α_6 is for the percentage of lapped reinforcement (not applicable for end anchorages)
- $l_{b,min}$ is the minimum anchorage length which is multiplied with the factor

$1.0 \leq \alpha_{lb} \leq 1.5$ that takes into account the product dependent sensitivity to cracked concrete of a post-installed rebar system as reported in the European Technical Assessment (ETA).

$$l_{b,rqd} = \frac{\phi}{4} \cdot \frac{\sigma_{sd}}{f_{bd,PIR}}$$

σ_{sd} is the tension stress to be anchored. In seismic applications this value is usually taken as the yield strength (f_{yd}) multiplied by an overstrength factor $\gamma_{Rd} \geq 1.0$ according to EN 1998-1.

$f_{bd,PIR}=k_b \cdot f_{bd} \leq f_{bd}$ (f_{bd} according to EN 1992-1-1 [7] and $0.7 \leq k_b \leq 1.0$ according to the relevant ETA) is the bond strength of the post-installed rebar system. Note that this value must be replaced by $f_{bd,seis}$ in seismic applications. Both $f_{bd,PIR}$ and $f_{bd,seis} \leq f_{bd,PIR}$ are reported from the relevant ETA for a specific rebar diameter, concrete strength class and drilling method.

Summarizing, according to EN 1992-1-1 [7], increasing concrete confinement results in utilizing the higher bond strength resulting in decrease in anchorage length until pull-out is reached (confinement of $c_d = 3\phi$) (Fig. 11). According to this approach, the design adequacy is checked for splitting (formation of radial cracks due to exceeding of tensile strength of the concrete around the rebar due to small cover or spacing), pull-out of rebar via shearing off concrete between the ribs and yield strength of the reinforcing bar (limiting the capacity of the connection).

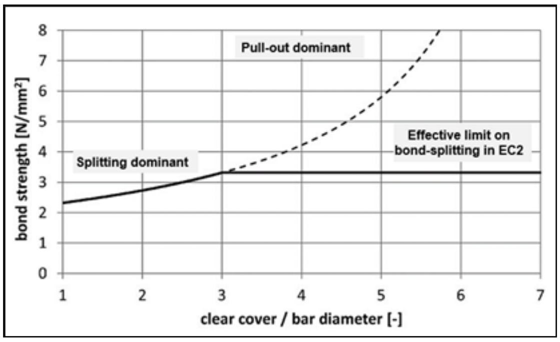


Fig. 11 Effective limit on bond splitting resistance as per EN 1992-1-1 [7]

Design accounting for specific post-installed rebar product performance

The introduction of EAD 332402 [13], [14] and [15] has established a comprehensive assessment of the product dependent bond-strength of post-installed rebars. A product with an ETA according to this EAD can be used for a design of end anchorages according to the EOTA Technical Report (TR) 069 [19]. The TR 069 [19]

Table 1. Values of coefficients α_1 to α_6 for cast-in as per EN 1992-1-1 [7] and post-installed rebars qualified as per EAD 330087 [6]

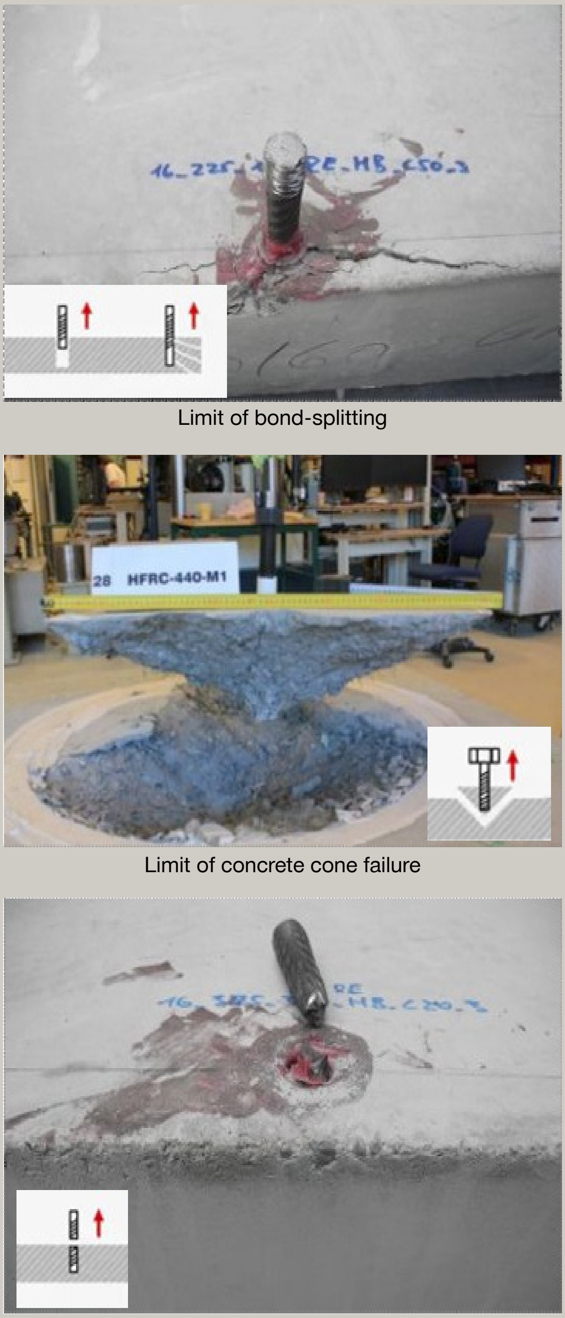
FACTOR	TYPE OF ANCHORAGE	CAST-IN REBAR		POST-INSTALLED REBAR	
		TENSION	COMPRESSION	TENSION	COMPRESSION
Shape of bar	Straight	$\alpha_1 = 1.0$	$\alpha_1=1.0$	$\alpha_1 = 1.0$	$\alpha_1=1.0$
	Hooked, bends	$\alpha_1 = 0.7$	$\alpha_1=1.0$	$\alpha_1 = 1.0$	$\alpha_1=1.0$
Concrete cover	All types	$0.7 \leq \alpha_2 \leq 1.0$	$\alpha_2=1.0$	$0.7 \leq \alpha_2 \leq 1.0$	$\alpha_2=1.0$
Confinement by transverse reinforcement	All types	$0.7 \leq \alpha_3 \leq 1.0$	$\alpha_3=1.0$	$0.7 \leq \alpha_3 \leq 1.0$	$\alpha_3=1.0$
Welded reinforcement	All types	$\alpha_4 = 0.7$	$\alpha_4=0.7$	$\alpha_4 = 1.0$	$\alpha_4=1.0$
Confinement by transverse pressure	All types	$0.7 \leq \alpha_5 \leq 1.0$	$\alpha_5=1.0$	$0.7 \leq \alpha_5 \leq 1.0$	$\alpha_5=1.0$
Percentage of lapped bars in the critical section	All types	$1.0 \leq \alpha_6 \leq 1.5$	$1.0 \leq \alpha_6 \leq 1.5$	$1.0 \leq \alpha_6 \leq 1.5$	$1.0 \leq \alpha_6 \leq 1.5$

Note: $\alpha_2 \cdot \alpha_3 \cdot \alpha_5 \geq 0.7$

is a guideline that includes provisions for the design of anchorages with post-installed rebars in moment resisting connections accounting for the product dependent bond-splitting performance.

The design as per TR 069 [19] follows the logic of limit state design. The approach is based on the establishment of a hierarchy of strengths between steel yielding ($N_{rd,y}$), concrete breakout ($N_{Rd,c}$), and bond-splitting ($N_{Rd,sp}$) (Fig. 13).

$$N_{Rd} = min\ (N_{Rd,y}, N_{Rd,c}, N_{Rd,sp})$$



Limit of bond-splitting

Limit of concrete cone failure

Limit of bar yeilding

Fig. 13: Failure modes as per TR 069 [19] design

The design yield resistance of the tension reinforcing bars ($N_{Rd,y}$) is calculated as follows –

$$N_{Rd,y} = f_{yk} \cdot A_s / \gamma_s$$

Where, A_s is the cross sectional area of tensioned reinforcing bars, f_{yk} is the characteristic steel yielding strength; γ_s is the steel partial factor.

For the calculation of the design concrete breakout resistance ($N_{Rd,c}$), the provisions of EN 1992-4 [20] are followed –

$$N_{Rd,c} = \frac{N^0_{Rk,c} \cdot \frac{A_{cN}}{A^0_{cN}} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N} \cdot \psi_{MN}}{\gamma_{mc}}$$

$$N^0_{Rk,c} = k_1 \cdot f_{ck}^{0.5} \cdot l_b^{1.5}$$

where: k_1 =7.7 or 11.0 for cracked or uncracked concrete, respectively, f_{ck} is the characteristic concrete compressive strength; l_b is the anchorage length of the reinforcing bar.

A_{cN}/A^0_{cN} takes into account the geometric effect of axial spacing and edge distance, $\psi_{s,N}$ is the factor for the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the concrete member, $\psi_{re,N}$ is the factor for the effect of dense reinforcement, $\psi_{ec,N}$ considers the load eccentricity and ψ_{MN} is the positive effect of a compression force in case of bending moments, with or without axial force.

The design bond-splitting resistance ($N_{Rd,sp}$) is calculated by considering a uniform bond strength distribution and using the analytical formulation derived from the fib Model Code 2010 [16] and qualitatively shown in Fig. 12 to define the splitting strength $\tau_{Rk,sp}$ with its influencing parameters (concrete strength f_{ck}, bar diameter ∅, minimum cover c_d, maximum cover c_{max} as defined in the fib Model Code 2010 [16] and the anchorage length l_b)
The factor A_k and the exponents s_p1, s_p2, s_p3, s_p4 and lb1 are product dependent parameters to be taken from the relevant ETA.

$$\tau_{Rk,sp} = \eta_1 \cdot A_k \left(\frac{f_{ck}}{25}\right)^{sp1} \cdot \left(\frac{25}{\phi}\right)^{sp2} \cdot$$

$$\left[\left(\frac{c_d}{\phi}\right)^{sp3} \cdot \left(\frac{c_{max}}{c_d}\right)^{sp4} + k_m K_{tr}\right] \cdot \left(\frac{7\phi}{l_b}\right)^{lb1}$$

$$\tau_{Rk,sp} \leq (\tau_{Rk,ucr} \cdot \Omega_{cr} \cdot \psi_{sus}) \ for\ 7\phi \leq l_b \leq 20\phi$$

$$or\ \leq\ (\tau_{Rk,ucr} \cdot \left(\frac{20\phi}{l_b}\right)^{lb1} \cdot \Omega_{cr} . \psi_{sus})\ for\ l_b \geq 20\phi$$

$\tau_{Rk,ucr}$ is the pullout resistance of the system assessed according to the EAD 330499 [19]. The factors Ω_{cr} and ψ_{sus} quantify its sensitivity to cracked concrete (0.3 mm) and sustained load, respectively.

Seismic design considerations –

For seismic design, the verification follows

$N_{Rd,eq} = N_{Rd,y,eq} \leq min\ (\ N_{Rd\ ,c,eq}\ ;\ N_{Rd,sp,eq})$

This means that yielding of steel should always be reached before any other (brittle) failure modes. However, in many cases, concrete breakout or splitting failures govern and thus in such cases, it is left to the designer to accept either splitting or concrete breakout as decisive failure mode if the predicted plastic mechanism of the structural system is ductile at a demand level at which the connection with post-installed rebars designed is still elastic.

For steel yielding, $N_{Rk,y,eq} = \gamma_{Rd} \cdot N_{Rk,y}$

Where γ_{Rd} is the overstrength factor related to the level of ductility for which the connection is designed according to EN 1998-1[8].

For concrete breakout, the following shall be considered – $N_{Rd,c,eq} = \alpha_{eq} \cdot N_{Rd,c}$

α_{eq} = 1 if the width of crack is equal to 0.3 mm
α_{eq} = 0.85 if the width of the crack is greater than 0.3 mm

The reduction factor α_{eq} = 0.85 is in line with the provision of EN 1992-4 [20] for single anchors and hence considering the effect of large crack width. No additional reduction for rebar groups, because is unlikely that tension rebars will experience different crack widths. For static loading conditions, a crack width of 0.3 mm can be assumed for designing. However, for seismic loading conditions, the expected crack widths can exceed the crack width limits given by EN 1992-1-1 [7] and reach crack widths of up to 0.8 mm. The maximum expected crack width in a connection is strongly affected by the overall behavior of the structure and is influenced by several factors such as the deformability of the existing member, the geometry of the connection, the design assumptions and the structural detailing of reinforcement bars. Generally, larger cracks are associated with connections that are designed to undergo larger deformations during a seismic event. Note that 0.8 mm is the upper limit of a flexural crack width prior to cross-section plasticization according to EN 1992-4 [20].

Furthermore, the ratio between anchorage length and thickness of the existing member is taken into account allowing the assumption of smaller crack widths for cases where the anchorage length is extended to approximately the entire thickness of the member. In such situations, practically, part of the anchorage is located in the compression zone and, therefore, the average crack width can be considered being smaller.

The resistance corresponding to pull-out and splitting failure is calculated as follows –

$\tau_{Rk,sp,eq} = \alpha_{eq,sp} . \tau_{Rk,sp}$

$$\tau_{Rk,sp,eq} \leq (\tau_{Rk,ucr} \cdot \Omega_{cr,eq} . \alpha_{eq,p})\ for\ \ 7\phi \leq l_b \leq 20\phi$$

$$or\ \leq\ (\tau_{Rk,ucr} \cdot \left(\frac{20\phi}{l_b}\right)^{lb1} \Omega_{cr,eq} . \alpha_{eq,p})\ for\ l_b \geq 20\phi$$

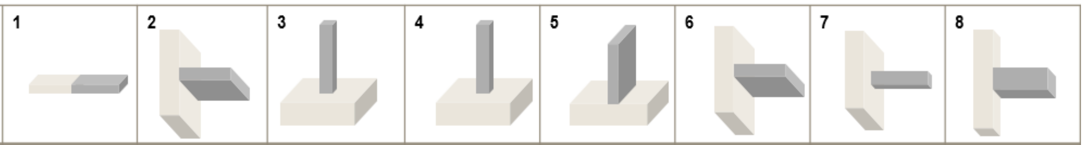
$\alpha_{eq,sp}$, $\Omega_{cr,eq}$ and $\alpha_{eq,p}$ are product dependent factors to be obtained from the ETA certificate of the post-installed rebar system assessed to resist seismic actions.

The splitting strength is reduced by the factor $\alpha_{eq,sp}$ due to seismic action accounting for the different energy dissipated in monotonic or cyclic loads. The factor $\alpha_{eq,p}$ accounts for the pull-out degradation due to cyclic loads and depends on the diameter. The parameter $\Omega_{cr,eq}$. varies with the rebar diameter and with the crack width.

CONCLUSION

There is a comprehensive set of guidelines available to design the post installed rebar connections, for static as well as seismic conditions. It is important to select the mortar whose performance has been assessed as per the relevant assessment documents and undertake a proper design of every connection.

A comprehensive overview is given in Fig. 14. Currently, in absence of adequate local design framework, there is inconsistency in the way these connections are treated. While a section of the designers exercise the right design practices, in many cases, the decision is made on prior experience, rule of thumb, generic specifications or random on-site pull-out tests. This leaves a lot of questions unanswered and compromises on the safety of the connection. It is pertinent to mention that the success of the entire retrofitting scheme, be it addition of shear walls or provision of concrete jackets, is to a great extent dependent on the performance of the connection and the ability of the post installed rebar to transfer the load as per design. An adequate anchorage depth based on the definite type of connection has to be determined for every connection and by no means, it can be generalized.



	1	2	3	4	5	6	7	8
Connection type	Splice	Simply supported (no moment)	Compression load only	Rigid	Rigid	Rigid	Rigid	Rigid
Members connected (examples)	Slab – Slab Wall - wall	Slab - wall	Column - foundation	Column to foundation	Wall to foundation	Slab to wall	Beam to wall	Beam to column
Design method	EC2			EOTA TR 069				

Fig. 14: Recommended design provisions for different type post-installed concrete-to-concrete connections

One may argue about the lack of local design guidelines, but this does not prevent us from adopting international design provisions which are well accepted and well researched. The need of the hour is make the structures safe against earthquake and efficiency of post installed rebars is a very crucial step in that.

REFERENCE

1. An Introduction to the Vulnerability Atlas of India, Third Version 2019, Building Materials and Technology Promotion Council, Ministry of Housing and Urban Affairs, Government of India
2. IS 15988:2013 Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines
3. Handbook on Repair and Rehabilitation of RCC Buildings published by Central Public Works Department, Government of India, Nirman Bhawan
4. Spieth, H. (2002). Tragverhalten und Bemessung von eingemörtelten Bewehrungsstäben (Behaviour and design of post-installed reinforcing bars, in German). PhD Thesis, University of Stuttgart.
5. Genesio et al. (2017): Genesio, G., Piccinin, R., Silva, J. (2017) Qualification of a system for post-installed reinforcing bars under the rules established by EOTA and ICC-ES, 3rd International Symposium on Connections between Steel and Concrete, Stuttgart, Germany, September 27-29, 2017.
6. EOTA EAD 330087 - 01 - 0601: Systems for post-installed rebar connections with mortar, December 2020, Brussel
7. EN 1992 - 1 - 1 : 2004 : Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings
8. EN 1998-1:2004: Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings
9. Simons, I.: Verbundverhalten von eingemörtelten Bewehrungsstäben unter zyklischer Beanspruchung (Bond behavior of post-installed reinforcing bars subjected to cyclic loading, in German). PhD Thesis, University of Stuttgart, 2007.
10. G. Genesio, R. Piccinin and S. Nerbano, Seismic Assessment of Post-Installed Reinforcing Bars with Mortar Based on the European Organization of Technical Assessment

11. Eligehausen, R., Popov, E. P. and Bertero, V. V. (1983). Local bond stress slip relationships of deformed bars under generalized excitation, UCB/EERC -83-23.
12. Rex, J., Sharma, A. and Hofmann, J. (2018). A new test specimen to investigate the bond behavior of post-installed reinforcing bars. Structural Concrete, 2018; 1-14, Wiley fib.
13. EOTA EAD 332402 - 01 - 0601: Post-Installed Reinforcing Bar (Rebar) Connections with Improved Bond-Splitting Behaviour Under Static Loading, October 2019, Brussel (adopted by EOTA, publication pending)
14. EOTA EAD 332402 - 01 - 0601 - v01: Post-Installed Reinforcing Bar (Rebar) Connections with Improved Bond-Splitting Behaviour Under Static Loading: 100 Years Working Life, August 2020, Brussel (adopted by EOTA, publication pending)
15. EOTA EAD 332402 - 01 - 0601 - v02: Post-Installed Reinforcing Bar (Rebar) Connections with Improved Bond-Splitting Behaviour Under Static Loading and Seismic Action, November 2021, Brussel (adopted by EOTA, publication pending)
16. Fib Model Code 2010 for Concrete Structures (2013), fédération internationale du beton/In-ternational Federation for Structural Concrete (fib), Lausanne.
17. EOTA EAD 330499 - 01 - 0601 (2018). Bonded fasteners for use in concrete, Brussel.
18. Cattaneo S, Genesio G, Piccinin R (2023) Seismic Design of Anchorages Using Post-installed Reinforcing Bars
19. EOTA TR 069: Design method for anchorage of post-installed reinforcing bars (rebars) with improved bond-splitting behaviour as compared to EN 1992-1-1, October 2019, Amended June 2021 Brussel
20. EN 1992-4 (2018): Eurocode 2: Design of Concrete Structures – Part 4: Design of fastenings for use in concrete, Brussel.

This webinar focused on steel buildings and structures that are rapidly gaining prominence in our country, particularly in various infrastructure segments. Even rooftop structures in existing buildings often utilize structural steel components. With the increasing frequency of earthquake tremors, understanding how to design these structures to be earthquake-resilient is paramount. Equally important is comprehending the behavior of connections in existing structures during earthquakes and addressing any issues that may arise.

The Seismic Academy, on behalf of Hilti, invited all civil engineering professionals and construction industry experts to an exclusive webinar. Participants gained insights into the following topics:

- Special design and detailing requirement for steel structures against earthquake
- Seismic design of connections using post installed anchors

To know more, click - https://theseismicacademy.com/webinar-detail/lets-explore-earthquake-resistant-steel-building-designs

DESIGN CONSIDERATION FOR AIRPORT BUILDING IN HIGH SEISMIC ZONE

Anamika Kadam
Deputy General Manager (Civil)
Larsen & Toubro

INTRODUCTION

The air travel market of India is one of the fastest growing in the world. With air passenger traffic expected to rise significantly and almost treble the numbers by in the next 20 years, the airlines are expanding their fleet size to meet the current and the projected traffic demand. This has resulted in the need to simultaneously expand and upgrade the airport infrastructure.

Fig. 1 Typical Airport Building

With such massive infrastructures being developed, it becomes equally important to ensure that these structures are also able to stand tall and unaffected within the most adverse natural hazards, that include earthquakes. In India, we are sitting in a seismically active zone, hence, adequate attention in this regard must be given while designing important service or community buildings like airports.

As per the current earthquake zonation in IS 1893-2016, the country's landmass is divided into 4 such zones namely Zone II to Zone V, depending on the severity of a possible earthquake basis historical data and extensive analysis by experts. The intent of this article is to provide a holistic view of the design approach that was adopted for the design of a lifeline structure like airport building located in high earthquake zone (e.g., Zone IV). Based on geotechnical investigation, the soil properties were evaluated and found to match the requirement of Type B (medium or stiff) soils in accordance with Table 1 of IS 1893.

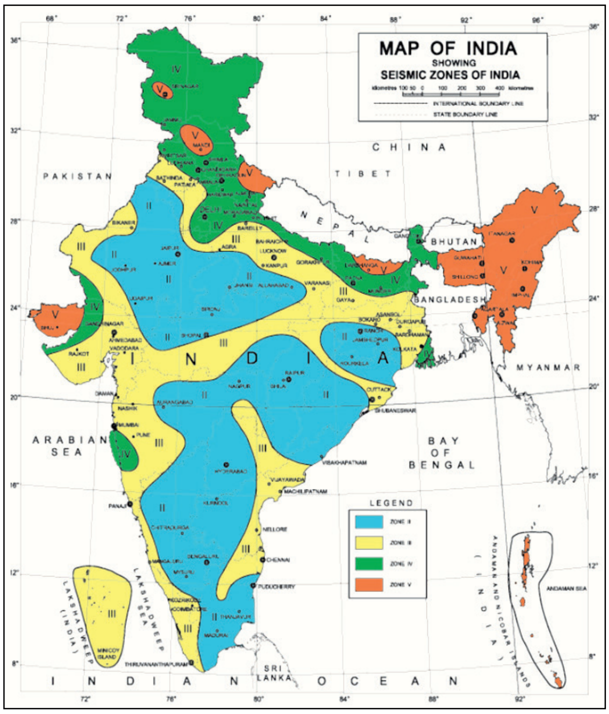


Fig. 2 Seismic Zones of India (Source IS 1893:2016)

DESIGN FOR STRUCTURAL MEMBERS –

To start with, the recommendations of the following codes and standards were strictly adhered to for design purposes–

- National Building Code of India 2016
- IS 875 for calculation design loads for buildings and structures
- IS 1893:2016 for determining the criteria of earthquake resistant design of structures
- IS 456:200 for design of reinforced concrete structures
- IS 13920:2016 for ductile design and detailing of reinforced concrete structures
- IS 800:2007 for general construction in steel.

The fundamental equation for calculation of horizontal acceleration coefficient was as per Cl. 6.4.2 of IS 1893 as follows –

$$A_h = \frac{(Z/2)\,(S_a/g)}{(R/I)}$$

The zone factor (Z) for zone IV was considered as 0.24 in line with Table 2 of IS 1893. For important buildings like airport the importance factor to be considered is 1.5 as per Table 3 if IS 1893. For ancillary buildings it was considered as 1.0. This has been resonated in Table 47 of NBC 2016 Volume 1. The response reduction factor for steel and RCC structures were taken as 5 in accordance with Table 1 of IS 1893.

Table 1. Response reduction factor (R) (Source IS 1893)

	LATERAL LOAD RESISTING SYSTEM	R
	Moment Frame Systems	
a)	RC buildings with ordinary moment resisting frame (OMRF) (see Note 1)	3.0
b)	RC buildings with special moment resisting frame (SMRF)	5.0
c)	Steel buildings with ordinary moment resisting frame (OMRF) (see Note 1)	3.0
d)	Steel buildings with special moment resisting frame (SMRF)	5.0

Table 2. Seismic Zone Factor (Z) (Source IS 1893)

SEISMIC ZONE FACTOR	II	III	IV	V
Z	0.10	0.16	0.24	0.36

Table 3. Importance Factor for buildings (I) (Source IS 1893)

SI NO.	STRUCTURE	I
i)	Important service and community buildings or structures (for example, critical governance buildings, schools), signature buildings, monument buildings, lifeline and emergency buildings (for example, hospital buildings, telephone exchange buildings, television station buildings, radio station buildings, bus station buildings, metro rail buildings and metro rail station buildings), railway stations, airports, food storage buildings (such as warehouses), fuel station buildings, power station buildings, and fire station buildings), and large community hall buildings (for example, cinema halls, shopping malls, assembly halls and subway stations)	1.5
ii)	Residential or commercial buildings [other than those listed in SI No. (i)] with occupancy more than 200 persons	1.2
iii)	All other buildings	1.0

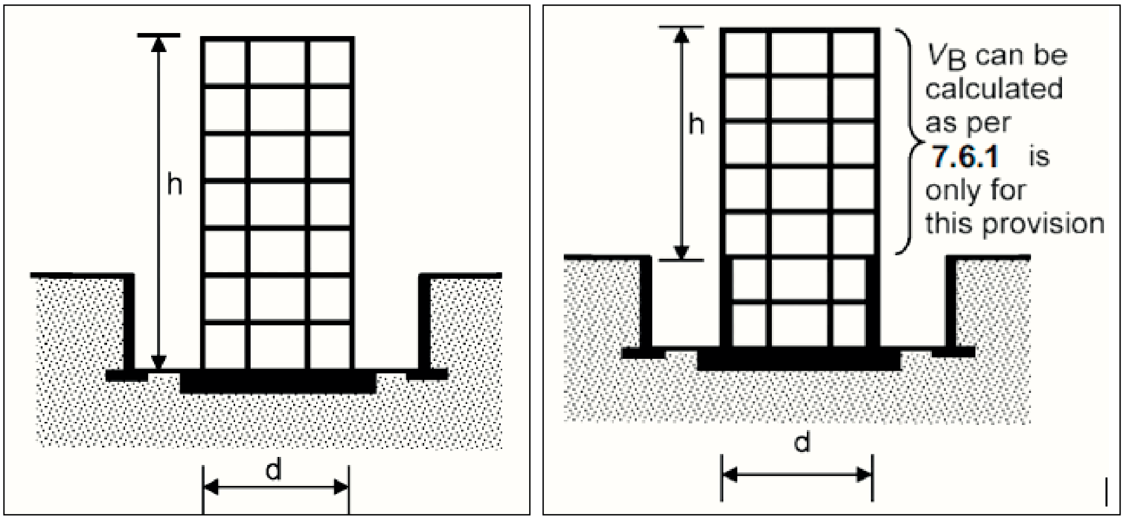
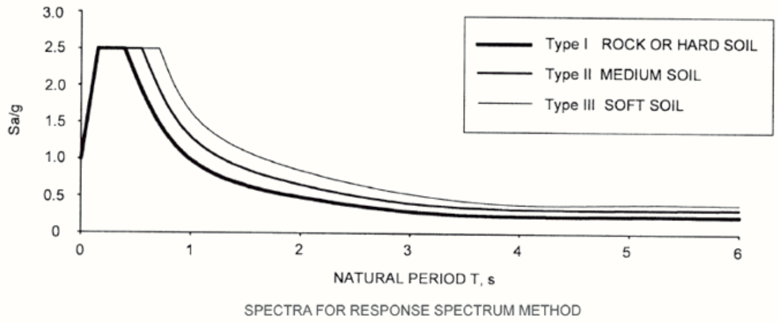


Fig. 3 Height of building for calculation of time period (Source IS 1893)



$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} \begin{cases} 1+15T & T < 0.10\text{ s} \\ 2.5 & 0.10\text{ s} < T < 0.40\text{ s} \\ \frac{1}{T} & 0.40\text{ s} < T < 4.00\text{ s} \\ 0.25 & T > 4.00\text{ s} \end{cases} \\ \text{For medium stiff soil sites} \begin{cases} 1+15T & T < 0.10\text{ s} \\ 2.5 & 0.10\text{ s} < T < 0.55\text{ s} \\ \frac{1.36}{T} & 0.55\text{ s} < T < 4.00\text{ s} \\ 0.34 & T > 4.00\text{ s} \end{cases} \\ \text{For soft soil sites} \begin{cases} 1+15T & T < 0.10\text{ s} \\ 2.5 & 0.10\text{ s} < T < 0.67\text{ s} \\ \frac{1.67}{T} & 0.67\text{ s} < T < 4.00\text{ s} \\ 0.42 & T > 4.00\text{ s} \end{cases} \end{cases}$$

Fig. 4 Design spectral coefficient for different soil types, corresponding to natural period of structure for response spectra method for 5% damping (Source IS 1893)

For calculations of fundamental time period of the structure were calculated as follows according to Cl. 7.6.2 of IS 1893 –

1). For RCC structure without masonry infill,
$T = 0.075\, h^{0.75}$

2). For RCC structure with masonry infill,
$T = 0.09h/\sqrt{d}$

3). For steel structure,
$T = 0.085\, h^{0.75}$

Where h = height of the building (in meters) and d = base dimension of the building at the plinth level along the considered direction of earthquake.

The vertical acceleration coefficient was calculated as per Cl. 6.4.6 of IS 1893 as follows –

$$A_h = \frac{(2/3 * Z/2)\,(2.5)}{(R/I)}$$

Table 4. Values of horizontal acceleration corresponding to time period for medium/stiff soil

PERIOD(S)	SA/G	ACC.	PERIOD(S)	SA/G	ACC.
0	1.00	0.353	2.00	0.68	0.240
0.10	2.50	0.883	2.10	0.65	0.229
0.20	2.50	0.883	2.20	0.62	0.218
0.30	2.50	0.883	2.30	0.59	0.209
0.40	2.50	0.883	2.40	0.57	0.200
0.50	2.50	0.883	2.50	0.54	0.192
0.55	2.50	0.883	2.60	0.52	0.185
0.67	2.03	0.717	2.70	0.50	0.178
0.70	1.94	0.686	2.80	0.49	0.172
0.80	1.70	0.600	2.90	0.47	0.166
0.90	1.51	0.534	3.00	0.45	0.160
1.00	1.36	0.480	3.10	0.44	0.155
1.10	1.24	0.437	3.20	0.43	0.150
1.20	1.13	0.400	3.30	0.41	0.146
1.30	1.05	0.369	3.40	0.40	0.141
1.40	0.97	0.343	3.50	0.39	0.137
1.50	0.91	0.320	3.60	0.38	0.133
1.60	0.85	0.300	3.70	0.37	0.130
1.70	0.80	0.283	3.80	0.36	0.126
1.80	0.76	0.267	3.90	0.35	0.123
1.90	0.72	0.253	4.00	0.34	0.120

Table 5. Values of vertical acceleration corresponding to time period for medium/ stiff soil		
PERIOD(S)	SALG	ACC.
0	2.50	0.589
0.50	2.50	0.589
1.00	2.50	0.589
1.50	2.50	0.589
2.00	2.50	0.589
2.50	2.50	0.589
3.00	2.50	0.589
3.50	2.50	0.589
4.00	2.50	0.589
4.50	2.50	0.589
5.00	2.50	0.589
5.50	2.50	0.589
6.00	2.50	0.589

The structure was designed taking into account three directional earthquake shaking in accordance with Cl. 6.3.4.1 of IS 1893, using the assumption that when the maximum response from one component occurs, the response of the other two components are 30 percent each of their maximum.

(i) $\pm EL_X \pm 0.3EL_Y \pm 0.3EL_Z$

(ii) $\pm EL_Y \pm 0.3EL_X \pm 0.3EL_Z$

(iii) $\pm EL_Z \pm 0.3EL_X \pm 0.3EL_Y$

The final load combinations were considered as follows –

(1) $1.2[DL + IL \pm (EL_X \pm 0.3EL_Y \pm 0.3EL_Z)]$

$1.2[DL + IL \pm (EL_Y \pm 0.3EL_X \pm 0.3EL_Z)]$

(2) $1.5[DL \pm (EL_X \pm 0.3EL_Y \pm 0.3EL_Z)]$

$1.5[DL \pm (EL_Y \pm 0.3EL_X \pm 0.3EL_Z)]$

(3) $0.9DL \pm 1.5(EL_X \pm 0.3EL_Y \pm 0.3EL_Z)$

$0.9DL \pm 1.5(EL_Y \pm 0.3EL_X \pm 0.3EL_Z)$

Dynamic 3D analysis was done for all the structures using finite element method. Response spectrum approach was used for the purpose.

The RCC detailing was done as per IS 456 and IS 13920. Ductile detailing was adopted for all RCC beams, columns, and walls.

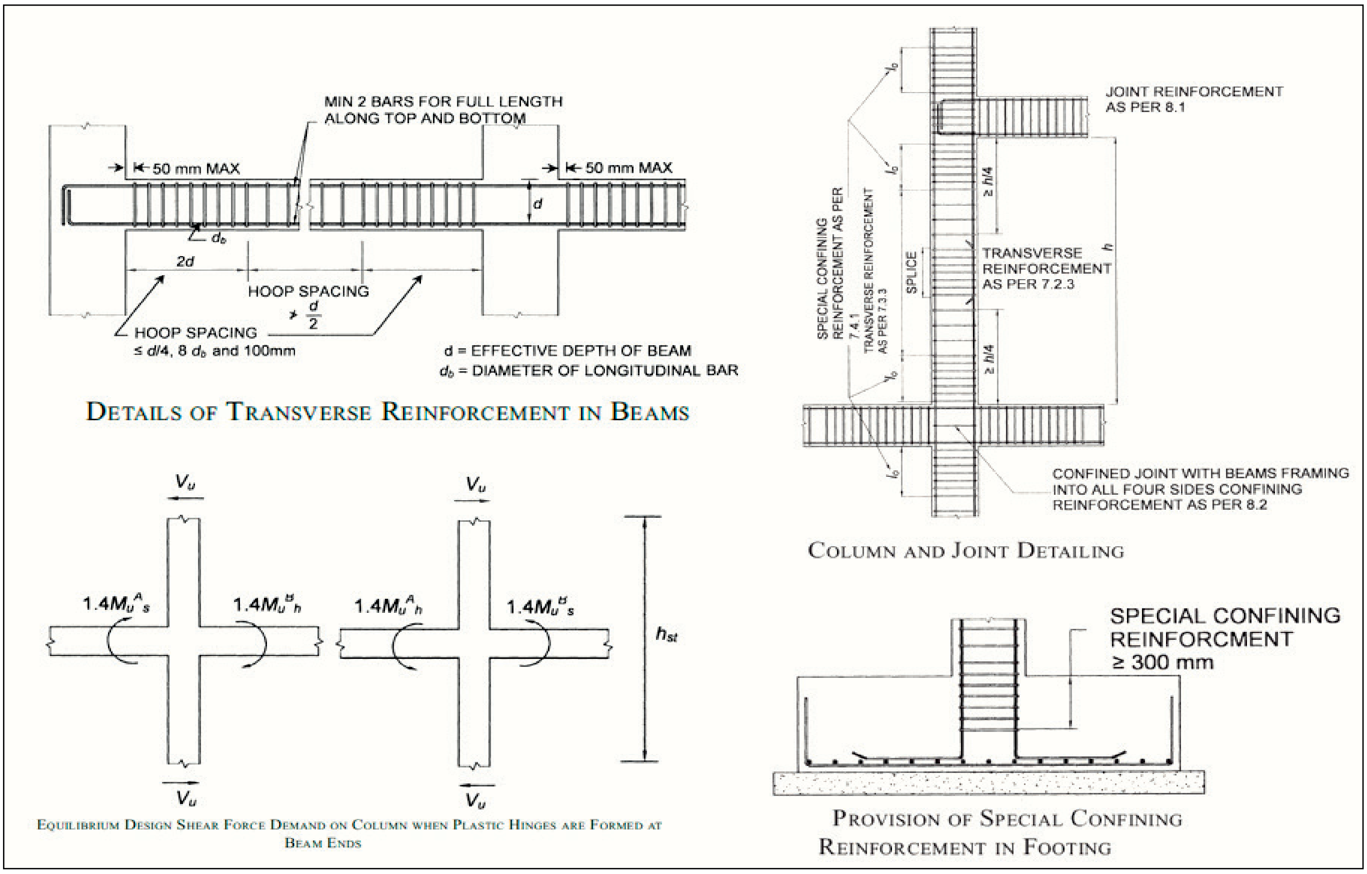


Fig. 5 Reinforcement detail in structural members (Source IS 13920)

For steel roof truss, normal connection details were adopted as per IS 800. For steel floor (e.g., mezzanines), ductile detailing as per section 12 of IS 800 was taken as the basis. Additional load combinations in accordance with section 12 of IS 800 were considered as follows –

(1) $1.2DL + 0.5LL + 2.5EL$

(2) $0.9DL + 2.5EL$

The sections selected for the beams and columns were checked to satisfy the following requirement as per IS 800 –

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.2$$

Where $\sum M_{pc}$ is the sum of the moment capacity of the column above and below the beam centerline and $\sum M_{pb}$ is the sum of the moment capacities of the beams at the intersection of beam and column intersection.

The individual thickness of the column webs and doubler plates –

$$t \geq (d_p + b_p)/90$$

Where t is the thickness of the column web or doubler plate, d_p is the panel zone depth between continuity plate and b_p is the panel zone width between the column flanges.

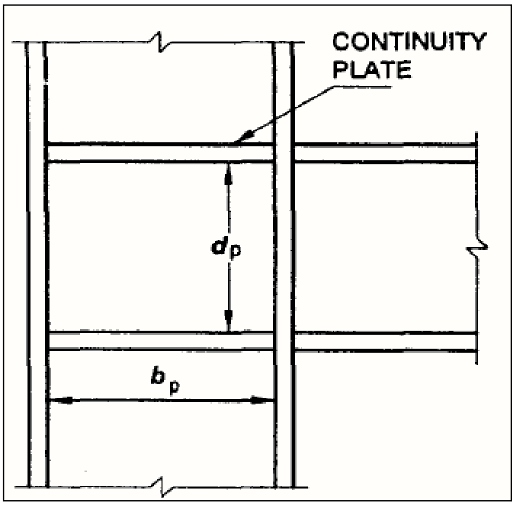


Fig. 6 Continuity plate

All beam to column connections were designed to withstand a moment of at least 1.2 times the plastic moment (M_p) of the connected beam. The connections were designed to withstand a shear resulting from the load combination of $1.2DL+0.5LL$ in addition to the shear resulting from the application of $1.2M_p$ in the same direction, at each end of the beam, resulting in double curvature.

All bolts used in frames designed to resist earthquake loads were fully tensioned high strength friction grip (HSFG) bolts or turned and fitted bolts. The welds used were complete penetration butt welds. The bolted joints were designed to ensure they did not share load in combination with welds on the same faying surface.

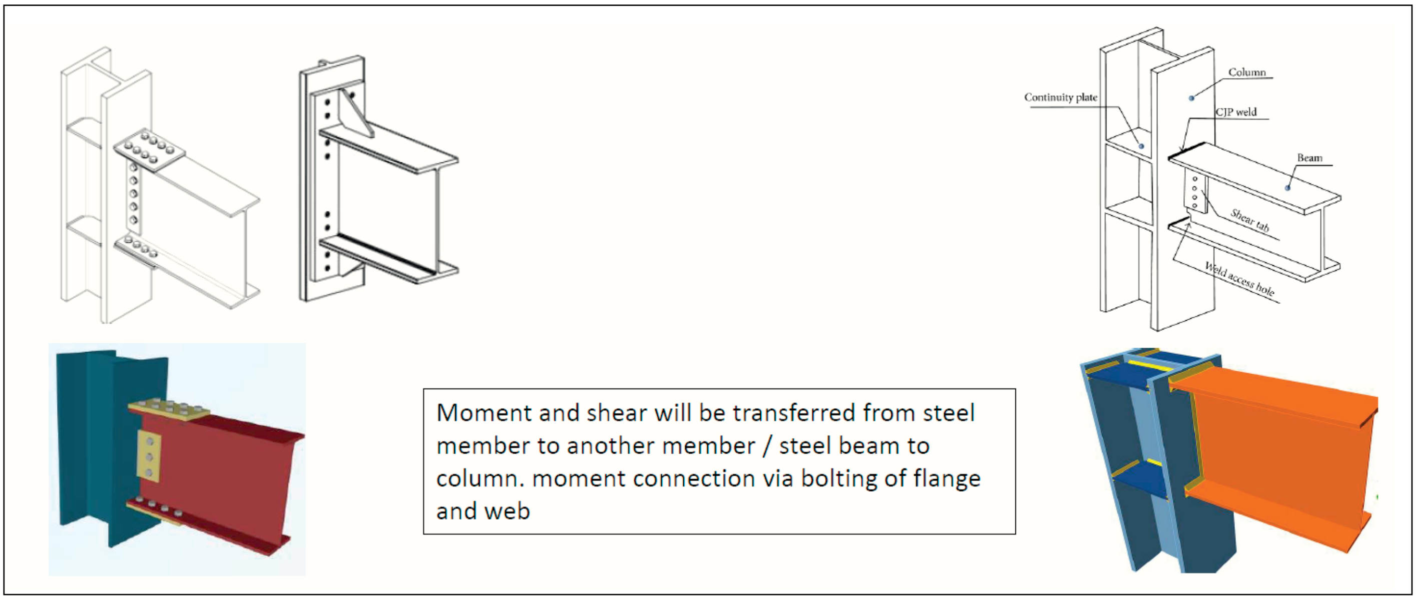


Fig. 7 Connection details

DESIGN FOR NON-STRUCTURAL ELEMENTS

Non-structural elements are extensively provided in airports to ensure buidling functionality. These are of paramount importance with respect to seamless operation and their failure in the event of an earthquake have severe repercussions which include loss of life, major damage to assets, completely jeopardising the safe evacuation and also can render the building non-functional. The National Building Code clearly mentions that the non-structural elements critical to operability of essential facilities such as hospitals, airports, emergency response centers, data centers, buildings vital to national defence, etc. continue to operate following strong earthquake shaking.They should be adequately attached to the supporting structure so that earthquake shaking does not cause them to topple or fall. Reference to clause 12.6.4 of NBC Volume 2 may be made in this regard.

Fig. 8 Typical arrangement of utility system in an airport building

In order to ensure the safety of the non-structural elements, the MEP supports were designed with a conservative approach considering zone V for the purpose. The support spacings and their arrangement were designed accordingly to render effective stability during an earthquake. It is important to not just secure the arrangements in place, but also avoid excessive sway since they may have a pounding effect on the adjacent supports and result in progressive failure.

CONCLUSION

The intent of this article is to present a guideline on codal reference which was adopted for design of an airport project in high seismic zone. The latest revisions of all relevant standards were adopted for the purpose of designing. However, the development in terms of evolution of standards is dynamic and the design work may be improved in future projects by adoption of more relevant standards like IS 18168 for Earthquake Reisstant Design and Detailing of Steel Buildings, IS 16700 for seismic design of non-structural elements, etc. In areas where we are still making progress with respect to development of standards, the encouragement is to adopt international standards to ensure comprehensive design is performed for each and every element of the structure. Any engineering judgement arising out of inadequate knowhow and thumb rule adoption, specifically in such critical structures, can really translate into major failure in adverse conditions.

STEEL CONCRETE COMPOSITE HIGH-RISE BUILDING WITH STEPPED ARCHITECTURE FOR EARTHQUAKE PRONE AREAS

Dr. Arup Saha Chaudhuri
Ph.D (Structure) IIT Bombay, F.IAStructE
Professor, Dept. of Civil Engg, Techno Main
Salt Lake, Kolkata

In structural engineering, steel-concrete composite structures are those types of structures where we use these two materials efficiently in the construction. They act as a single unit in which steel is effective on tension side and concrete is effective on compression side. In this paper, the proposed 22 storey high-rise building frame is made of structural steel columns and steel beams with concrete slab composite floors. Stepped Architecture is one of the ideal concepts of construction to stabilize any framed structure. The bottom portion should be much wider than the top portion of the structure in this concept.

This concept is very suitable for high-rise buildings in earthquake prone areas. We will discuss about composite construction and stepped architecture concepts in detail and will show how we can apply both of these concepts in high-rise building to work efficiently. Purpose of this paper is to design and analyze a two dimensional building frame under high seismic zone without providing any extra seismic resisting system.

Keywords:
High-rise steel building, Earthquake prone area, Stepped architecture, Vertical stability bracings, Steel-concrete composite floors.

INTRODUCTION

We all know that 71% of earth surface is covered by water and remaining part is covered by land. Population of world is increasing day by day but our land of earth is limited. So it is not possible to built house for each and every individual person.

In our modern days of civilization, construction of tall buildings is rapidly increasing where maximum person can live by using minimum space of land. This tall building is constructed not only for residential purpose but also can be used for commercial purpose or both. There is no such definition of tall or high-rise building. But as per IS Code RC buildings of height more than 50 m but less than 250 m can be treated as a tall building but this standard is not applicable for location of building near field of seismogenic fault.

Composite is that where two or more materials or units of different properties are combined together and these materials or units act as a single unit. Composite construction is widely used method in modern days of constructions. Scientists are doing research on this theory that how to develop more composite construction in different ways. Engineers are also adopting this technique in construction industries. Composite construction is widely used in building construction, aircraft and watercraft.

There are some examples of composite construction like – Steel-Concrete composite deck, Wood-Plastic composite deck, Cement-Polymer composite etc. Composite constructions have some advantages like high strength, high stiffness, high seismic resistance, increased load carrying capacity, economic, lightweight and environment sustainability.

> **“Stepped Architecture is one of the ideal concepts of construction to stabilize any framed structure.”**

Most of the high-rise buildings have more tend to experience prolonged shaking than short buildings because they often have lower damping and body waves from earth rapidly travels through the ground compared to slower, more destructive wave. They are not safe enough to resist vibrations. Hence, tall buildings are not safe against earthquake. It has major chance to damage of properties and lots of life loss. Tall buildings are not safe even in Zone – II. For example, we can say about 2001 Bhuj earthquake where high-rise buildings of Ahmedabad city were damaged epicenter was 300 km away from it.

To resist the affects of earthquake we have to apply some modern technologies by installing seismic isolation devices. These devices reduce the energy of structure and reduce forces acting on floors. These devices increase the stiffness of structures and also increase the capacity of structures to resist loads. There are so many devices those can be used as per the design like Synthetic Rubber Bearing or Lead Rubber Bearing, Fluid or Viscous Dampers, Visco-Elastic Damper, Rocker Roller etc. Sometimes we can use some design concept for earthquake resistance building like Shear Wall concept, Braced Frame concept etc.

There is a lot of research on the best shapes for earthquake resistance buildings. Buildings can be irregular or asymmetrical in shape. Some shapes those have been found to perform well in earthquake include Triangular shape, Rectangular shape, Dome shape, Stepped shape etc. In this paper, we will focus on stepped shape with no extra seismic resistant mechanism.

STEEL CONCRETE COMPOSITE

Steel-Concrete composite is one of the most widely used among all composite structures. This type of composite slab is generally used in bridges and multi-storey buildings. Because of composite action, it has higher stiffness, higher strength, higher span to depth ratio, lower deflection than traditional steel or concrete. Concrete is strong in compression where steel is strong in tension. Therefore, it is proven that steel-concrete composite enhances the structural performance.

> “Steel-Concrete composite is one of the most widely used among all composite structures.”

Composite deck is a combination of the compressive strength of concrete with the tensile strength of steel to improve the design efficiency and potentially reduce the volume of material necessary to cover a given area. A profiled sheet of metal supported by steel joist or beam is the shuttering cum reinforcement. Then fresh concrete is poured on top of this sheet and it becomes a composite deck. The advantage of using composite deck is the increased strength of the floor without adding any extra weight.

Due to high load carrying capacity, larger span, high diaphragm action, easy installation process, minimal wastage and good safety for workers composite deck is proposed by the designers now-a-days.

EARTHQUAKE RESISTANT BUILDING

Releases of energy due to movement of tectonic plates huge damages occur in the structures like tall buildings, bridges etc. The tall buildings are more flexible than the short buildings so it has more chances to damage by earthquake. This is so destructive that is enough to kill lot of people and massive loss of economy. Hence, seismic analysis is very much needful for tall buildings. In our country, we have all four seismic zones i.e.

Zone 2 to Zone 5. This analysis is followed by IS codes and depends on earthquake zones, soil strata, type of structure, seismic weight of building, ground acceleration etc. Effects of design earthquake loads applied on structures can be considered in different analysis method such as equivalent static method, response spectrum method etc. Various methods of earthquake resisting systems are also applied like shear wall, core wall, braced frame, base isolation, different types of dampers etc.

OUR CASE STUDY

Only steel or concrete buildings both have drawback in wind and earthquake respectively. According to our study to improve the properties of tall buildings we should use steel concrete composite. A study says that composite systems are over 25% lighter than concrete construction.

In high-rise building seismic reaction affects horizontally and tortionally. In general bracing and shear wall are to be designed for stiffness because bracing and shear wall aim to dissipate this poor seismic behavior.

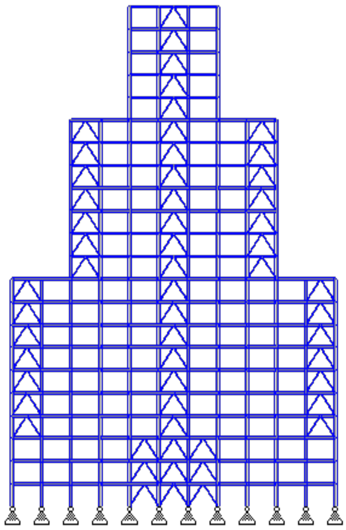
According to the study, we noticed that storey displacement is more in symmetrical building, so we propose to design different architectural concept that is horizontally or vertically irregular in shape. Triangular or pyramidal shape is more prominent for earthquake resistant building. So in this case we design a building which is cascade shape or also we can say this type of architecture is stepped architecture. All over the world we can see the concept of stepped architecture. Shenye Tairan Building (Shenzhen City, China) and Aspern J4 (Vienna, Austria) are the examples of among stepped architecture.

STAAD-PRO software is used for seismic analysis of buildings. The result shows that bracings are much more efficient than shear wall in reducing lateral displacement of frame as drift and horizontal deflection are much less than shear wall. Column axial forces are more in braced frame than shear wall and column & beam moment is less than shear wall. CCTV Headquaters (Beijing, China) and Hearst Building (New York City, USA) are the best example of tall building with bracing.

> **“Triangular or pyramidal shape is more prominent for earthquake resistant building.”**

DESCRIPTION OF STRUCTURE

In this paper, a 22-storey residential two dimensional building frame is considered which is to be designed under seismic loading. The building shape has three steps. The first step is constructed from the Ground floor to 10th floor and also has 11 bays with distance of 5 m each. The second step is designed from 11th floor to 17th floor. We reduce two bays from all sides of the frames of second step and also has 7 bays with distance of 5 m each. Similarly, the third step again we reduce two more bays from all sides of frames. This third step is 18th floor to top of the building frame. Each floor height of this building is 3.25 m.



The building is made of steel-concrete composite by using wide flange steel beam & column sections (UB/WPB) and steel bracing sections (SHS/RHS). Consider the building is located at Zone IV in India. From IS 1893 (Part-1):2016, Table-3 we get Seismic zone factor Z is 0.24. The response reduction factor R is 4. Importance factor I is 1.2 as per IS code.

The soil strata of construction site is assumed as medium stiff. For this design horizontal seismic co-efficient is calculated Ah as 0.036. We provided damping 5% on this building frame. The dead load is considered as 5 kN/sq.m including its self weight for all floors. The live load is considered as 4 kN/sq.m from 1st floor to 5th floor and 3 kN/sq.m from 6th floor to 10th floor on the first step.

On the second and final steps we considered live load 2.5 kN/sq.m. The nodal load is 67.5 kN for all nodes at the edge of the building. Water tank load is considered at the roof of each step. The seismic load is acting towards horizontal direction on the building frame.

We have analyzed the model by response spectrum method. The set of load combinations involving seismic effects are as follows:

1. DL + LL
2. DL + LL + EQL
3. DL + LL – EQL
4. 1.5 (DL + LL)
5. 1.2 (DL + LL + EQL)
6. 1.2 (DL + LL - EQL)

Temperature stress analysis should also be carried out and proper structural arrangements for releasing the temperature stress must be implemented in the main structure.

The building is then suitably designed in STAAD Pro software using response spectrum method. We can see the mass participation factors of our building frame in the following table which is given below:

Mass Participation Factors

	MASS PARTICIPATION FACTORS IN PERCENT						BASE SHEAR IN KN		
MODE	X	Y	Z	SUMM-X	SUMM-Y	SUMM-Z	X	Y	Z
1	58.77	0.00	0.00	58.765	0.000	0.000	435.69	0.00	0.00
2	22.67	0.00	0.00	81.434	0.000	0.000	399.63	0.00	0.00
3	7.30	0.00	0.00	88.737	0.000	0.000	241.95	0.00	0.00
4	6.71	0.00	0.00	95.443	0.000	0.000	239.85	0.00	0.00
5	0.00	50.40	0.00	95.443	50.401	0.000	0.00	0.00	0.00
6	0.00	0.02	0.00	95.445	50.420	0.000	0.08	0.00	0.00
				TOTAL SRSS		SHEAR	682.34	0.00	0.00
				TOTAL 10PCT		SHEAR	682.34	0.00	0.00
				TOTAL ABS		SHEAR	1317.19	0.00	0.00
				TOTAL CSM		SHEAR	682.34	0.00	0.00
				TOTAL CQC		SHEAR	694.34	0.00	0.00

Fundamental Time Periods And Modal Base Actions

MODAL BASE ACTIONS FORCES IN KN LENGTH IN METE

MODE	PERIOD	FX	FY	FZ	MX	MY	MZ
					MOMENTS ARE ABOUT THE ORIGIN		
1	2.624	435.69	0.11	0.00	0.00	0.00	-19826.53
2	1.104	399.63	-0.36	0.00	0.00	0.00	-4361.20
3	0.587	241.95	-0.35	0.00	0.00	0.00	-1387.85
4	0.411	239.85	1.67	0.00	0.00	0.00	-935.67
5	0.330	0.00	-0.91	0.00	0.00	0.00	-24.92
6	0.302	0.08	-0.23	0.00	0.00	0.00	117.98

DEFLECTION CHECK

FLOOR LEVELS	TIP HORIZONTAL DEFLECTION (MM)	HEIGHT / 500 (MM)	REMARKS
Above 10th floor	25	65	DL+LL+EQL
Above 17th floor	46	110	DL+LL+EQL
Above 22nd floor	65	143	DL+LL+EQL

CONCLUSION

We designed and analyzed the building for seismic zone – IV by response spectrum method. Without any special seismic resistant systems like – Shear wall, Damper bracings etc. this design is totally safe implementing stepped architecture design considerations with stability bracings.

REFERENCES

1. Tall Building Design – Steel, Concrete and Composite Systems by Bangale S. Taranath.
2. IS 1893 (Part 1): 2016 Criteria for Earthquake Resistant Design of Structures.
3. IS 800: 2007 Steel structure design code of practice.
4. Literature survey on stepped architecture concept in the internet.

NEW WILSHIRE GRAND CENTER (LOS ANGELES, CALIFORNIA) – Where Innovation Meets Resilience

The Wilshire Grand Center, a prominent 73 storey structure, situated in downtown Los Angeles, has reshaped the western skyline of downtown Los Angeles and stands as a testament to engineering marvel and finesse. Encompassing around 2 million square feet, this structure seamlessly integrates hotel and office spaces. While the top of the structure features restaurants and a sky lobby overlooking the skyline, the surrounding podium structure include additional commercial spaces for a burgeoning resurgence of the surrounding area. The building was the brainchild of Architect AC Martin Partners Inc while M/s Brandow and Johnson were bestowed to fortify the structure against all natural hazards. M/s Thornton Tomasetti was involved in the performance based analysis and design and M/s Turner Construction Company was responsible to breathe life into the structure.

Details

The precise architectural details including its 100 feet crown sail designed to emulate the Half Dome in Yosemite National Park and a 30 feet tall LED-laced spire it dramatically redefines the Los Angeles skyline as the city's only building without a flat top roof. It was built into a thick foundation made from the largest continuous concrete pour in history dumping 82 million pounds. It was then supplemented with buckle-resistant braces (BRBs) at levels 70-73, that would act as shock absorbers in the case of an earthquake or strong wind. The structure was built with almost 19000 tons of structural steel.

The structural steel-framed tower is geometrically complex, with many of the steel columns sloping over the height of the building to ensure the curved periphery. Between the 28th and 30th floors, the exterior building columns slope inward 6 feet over the three floors to transition the floor plate configuration

"The structural steel-framed tower is geometrically complex, with many of the steel columns sloping over the height of the building to ensure the curved periphery."

from office to hotel space. The columns are embedded the full depth of the 18-ft-thick concrete mat foundation to anchor seismic uplift forces. The design team implemented a performance-based design methodology to accommodate the utility requirement.

A conventional code-prescribed lateral design would have required a perimeter lateral system on the structure in addition to the concrete core wall, resulting in deep perimeter beams. This would have either increased the storey heights or reduced the heights of the window opening size. This would have increased the overall project duration. The building is designed to be linearly elastic for a service-level earthquake with a 43-year return period, and for collapse prevention for the extremely rare 2,475-year return period earthquake. To achieve this performance, the design team created three buckling-restrained brace (BRB) regions over the height of the structure. A total of 170 BRBs distribute lateral overturning forces to the exterior concrete-filled steel box columns.

Structural Analysis and simulation:

Before commencement of construction, AMEC who was the geotechnical consultant for the project, simulated earthquakes to validate the performance under extreme conditions. Working with data prepared by the California Geological Survey and the Southern California Earthquake Center, the analysis began by cataloging nearly 100 local faults, poring over analyses of their geometry, their type, their slip rate and maximum possible magnitude. The engineers studied how waves of energy, generated by earthquakes ranging from magnitude 4 to the low 8s, moved through the earth across Southern California and extrapolated how the earth movements would translate into shaking. With the help of an independent review board, they culled through 3,551 recordings of 173 earthquakes taken by 1,456 monitoring stations around the world and came up with 11, the best representation of the most severe earthquakes the building would experience, based on historic data.

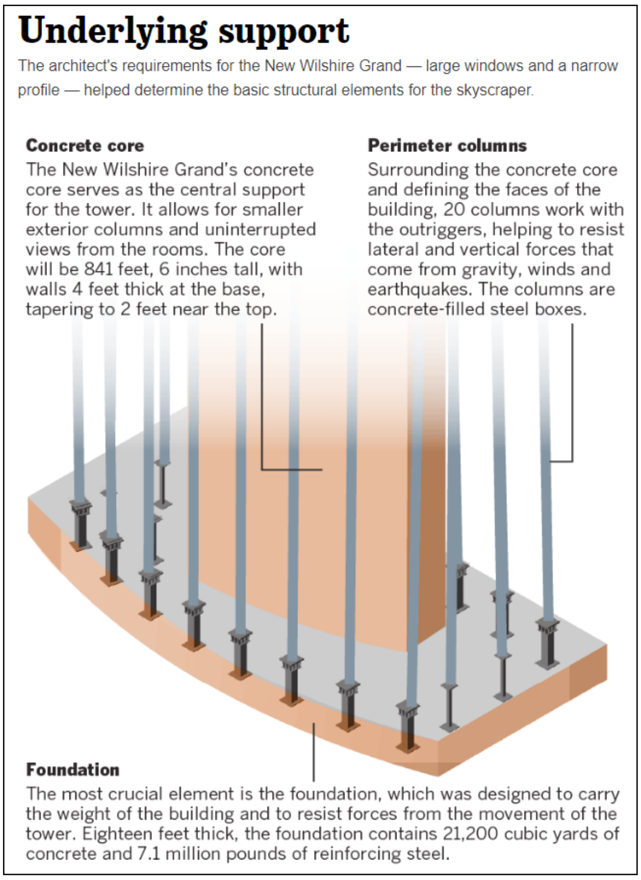


Fig. 1

With the data in hand, the next step was to test the information against the New Wilshire Grand's specifications. The tower was built around a concrete core that rises 841 feet and 6 inches thick. Its walls are 4 feet thick at the base and 2 feet near the top. The entire building weighs 300 million pounds. This required the engineers to work upon multiple data points (112,500 lines of information) that included information like the size and location of the beams, columns and walls, along with their strength, stiffness and behavior when overloaded. The team thoroughly scrutinized the data.

Based on the results of the tests, the engineers redesigned the size and depth of the foundation to resist a much as 13.2 million pounds of force pulling up and 25 million pounds of force pushing down on each of the 20 perimeter columns as the tower swayed during an earthquake.

The Wilshire Grand significantly features a seismic joint between the base and the tower that allows for 1.5 feet of sway without causing damage to partitions or pipework.

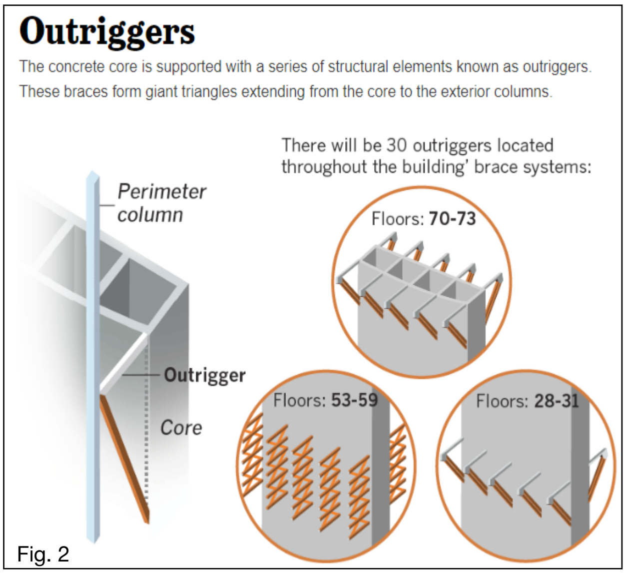


Fig. 2

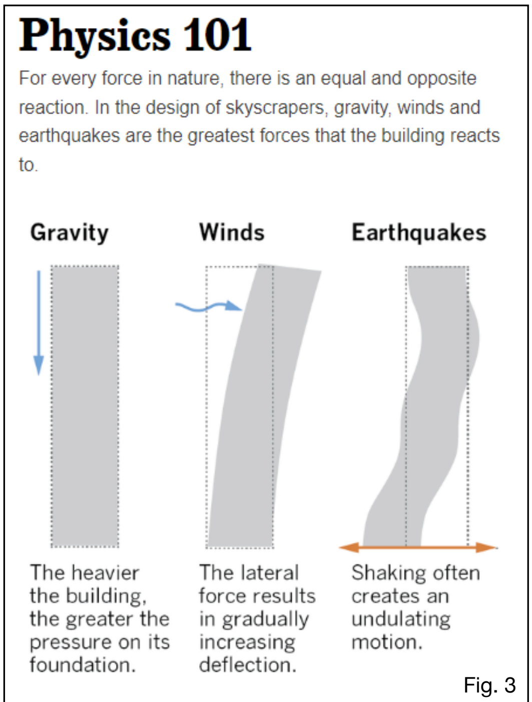


Fig. 3

The numbers pointed out a major problem. Strained by the force of earthquakes, the outriggers jammed into the core, delivering more stress than the concrete could absorb. The inside walls between the elevators and stairwells were failing. And this could lead to wide cracks forming in the core. Initially the intent was to add more concrete to the walls, but that would crowd the elevator shafts. Placing steel plates inside the walls would slow the construction and raise costs. And this led to the recommendation to add BRBs. These devices are long steel bars encased in a steel box filled with grout that allows the bars to compress or stretch as the building moves. There are 170 of the BRBs used in the construction of the building. At the top of the structure there exist ten 2,200-kip BRBs extending from floors 70 to 73. Between floors 53 and 59 are 120 800-kip BRBs, with each spanning only one floor and hidden in the hotel room demising walls—a unique configuration that allowed the developer to maximize the hotel room count. Closer to the bottom of the structure, between floors 28 to 31, are 40 2,200-kip BRBs. Bundled in groups of four at ten locations, they span three floors and are capable of resisting 8,800 kips at each location. The extensive system of BRBs is complemented by perimeter belt trusses around the exterior between levels 28 and 31 and levels 70 and 73. These elements all work together to provide torsional resistance and load path redundancy.

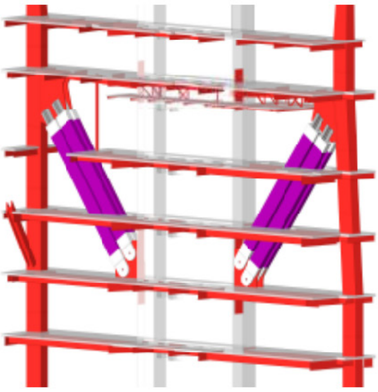


Fig. 4: Lower Outriggers with Double - Double BRBs


Fig. 5: Embed plate for lower

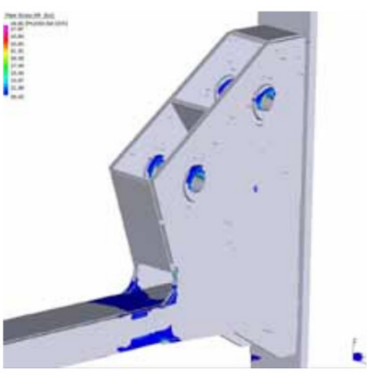


Fig. 6: FEA model for connection at core Courtesy of SIE, Inc.

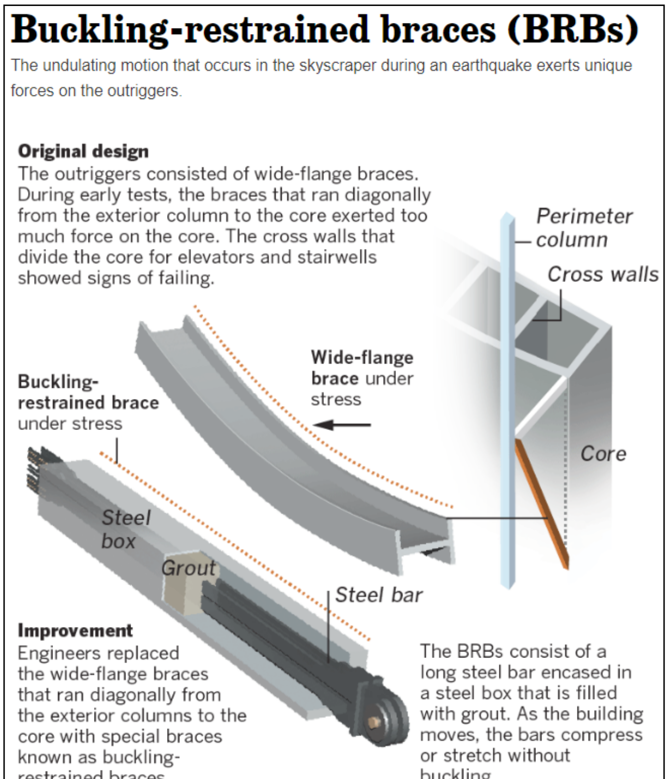


Fig. 7

The lower and the upper outriggers were connected to the core wall with steel embed plates. Shear studs and half-couplers were welded to the back of the embed plates to meet the desired force demands. The embed plates were up to 4 inches thick, stand over 34 feet tall. Gusset plates were welded to the embed plates to receive the double-pinned connections for the double BRBs. The sensitivity of concrete to heat from the welding of the gusset plates led to the use of electroslag welding with tight tolerances of 3/8 inch for horizontal control of the embedment plate.

One of the challenges in designing the Middle Outriggers was the need to accommodate a large "notch" in the outrigger girders adjacent to the core wall for mechanical, electrical and plumbing utilities. Due to the short floor-to-floor height, the notches were required to provide a path for ducts, conduits and pipes. The outrigger girders at these locations were heavy members connected to an embedded plate with a gusset plate and pin. Each girder was reinforced with plates to provide the required strength at the notch.

It was assumed that after completion of the structure, the elastic shortening of the steel would be complete except for that associated with occupant live loads. Due to the thickness of the concrete core walls, it would take approximately 50 years for 75% of the concrete shrinkage to occur. With shrinkage and creep of the core wall, the BRBs would go into tension. The Upper Outrigger BRBs were single 2,200-kip braces, sensitive to the differential movement between the shrinkage, creep and elastic shortening of the core wall and the elastic shortening of the structural steel box columns. A pre-compression force of 1,000 kips was used for each of the upper BRBs on alternate sides of the build to reduce tensile force between braces. Once the compressive strains reached approximately a 1/2 inch, the BRB was bolted off. A total of 500 tons of compression was jacked into the braces creating tension in the exterior building columns with each brace pushing upwards on the building's perimeter.

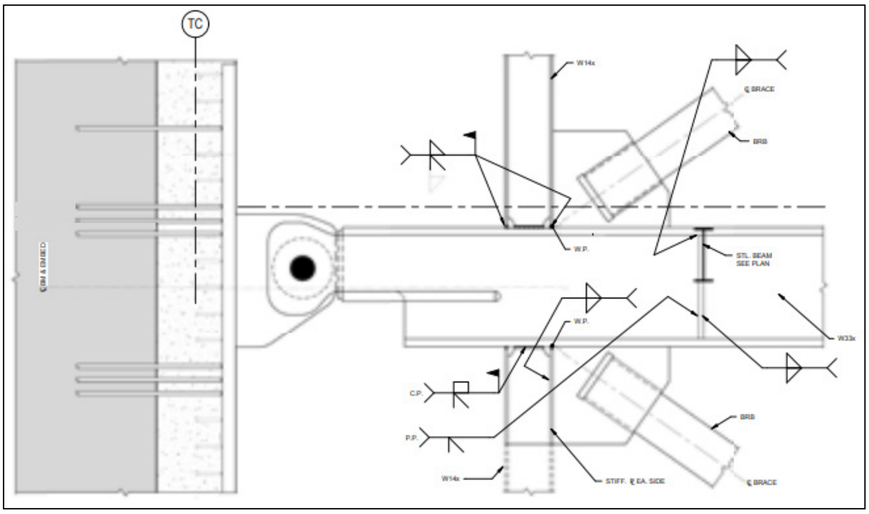


Fig. 8: Middle Outrigger girder connection to embed plate.

Movement at the base of the tower could amplify into a roller coaster ride at the top. With possible accelerations of 4g, engineers worried that the crown and spire might buckle. However, removing those architectural elements was not an option. Luminous by day, illuminated by night, the sail-like crown was the building's hood ornament, a distinctive mark in the city's skyline.

Engineers considered anchoring the sail to the building with long cables that would allow a gentle rocking. But further tests showed that the sail would rock so violently that it would damage the concrete core. A redesign of the sail into a shorter feature offered no advantage. The sail had to be made sturdier, less light and airy.

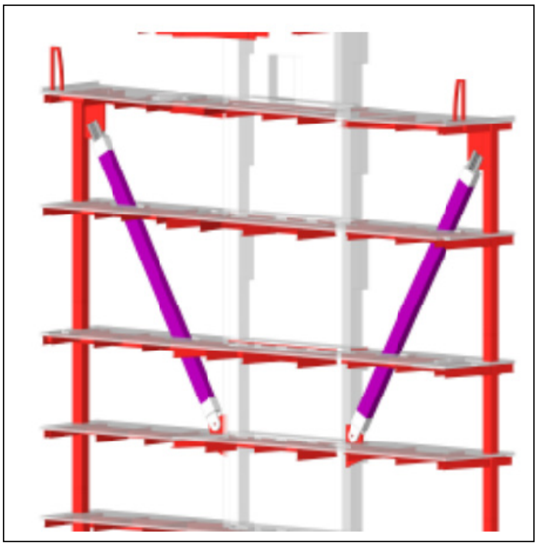


Fig. 9: Upper outrigger BRB

Conclusion

As the Wilshire Grand Center graces Los Angeles with its imposing presence, it sets a precedent for the seamless integration of technological sophistication with architectural splendour. This skyscraper stands tall, not just in height but as a symbol of cutting-edge design that prioritizes safety, particularly in regions prone to seismic activity.

REFERENCES

1. https://edition.cnn.com/style/article/quakeproof-skyscraper-new-wilshire/index.html
2. https://www.aisc.org/awards-and-competitions/ideas2-awards/ideas2-awards-archives/wilshire-grand-center/
3. https://conconow.wordpress.com/2016/09/21/sophisticated-earthquake-resistant-features-on-the-wilshire-grand/
4. https://www.structuremag.org/wp-content/uploads/2015/07/SF-WilshireGrand-Aug151.pdf
5. https://www.schuff.com/wp-content/uploads/2023/01/Wilshire_Case-Study.pdf
6. https://graphics.latimes.com/wilshire-grand-earthquakes/
7. https://www.wilshiregrandoffices.com/about/
8. https://www.sgh.com/project/wilshire-grand-center/
9. https://wilshiregrandcenter.com/about/
10. https://www.aisc.org/globalassets/cvsic/wilshire_grand_case_study.pdf

Vol. 05
March 2024

SEISMIC ACADEMY

“WE CANNOT STOP NATURAL DISASTERS BUT WE CAN ARM OURSELVES WITH KNOWLEDGE: So many lives wouldn’t have to be lost if there was enough disaster preparedness.”

- Petra Nemcova

FOR MORE DETAILS ON THIS INITIATIVE, PLEASE VISIT OUR WEBSITE
https://theseismicacademy.com