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Strengthening and
Retrofitting of Structures

Seismic Splendour
San Francisco City Hall

**Interactive
Crossword**



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To make the seismic academy a one-stop source of information and use it for the promotion of all seismic initiatives in our country.

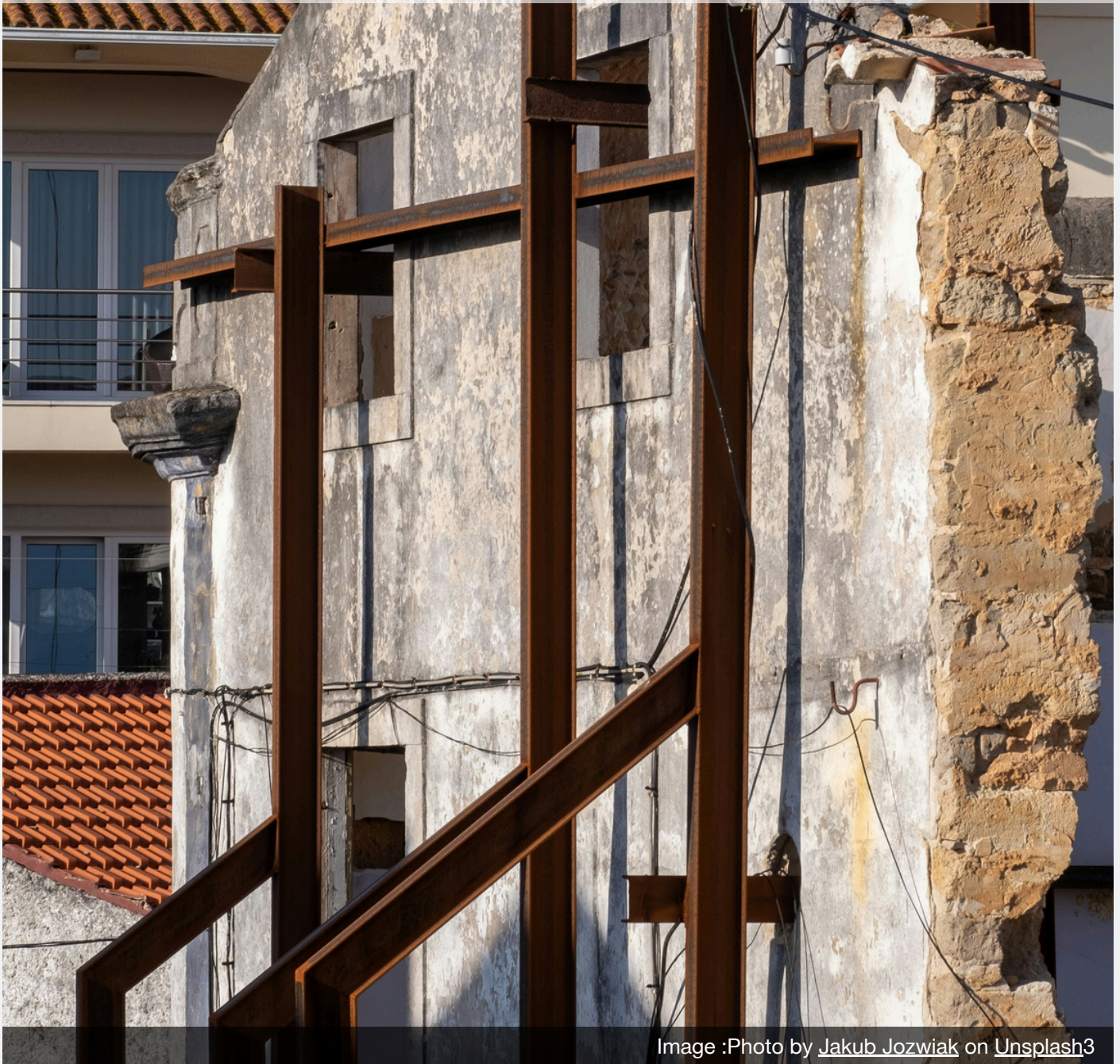


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Test your Seismic Engineering Knowledge



Kobe Earthquake 1995

“ Seismic strengthening is an investment in resilience, preserving the buildings we value while preparing them to withstand the forces we cannot predict. ”



Fa-bo: The Earthquake Resistent Office, Honshu, Japan




SPEAKER

Er. Jitendra Kumar Chaudhary
Scientist 'C'/Deputy Director
(CED 2 and CED 39, BIS)

SPEAKER

Er. Barry Kaknics
Member of the Board and
Technical Committee, AEFAC

MODERATOR

Agamoni Das
Head - Codes & Approvals
Fastening, Hilti India

WEBINAR

May 21st, 2026
11:30-1:00PM (IST)

Seismic Design of Structures and Connections

Evolution of India's Seismic Design Framework

Opening the technical discussions, Er. Jitendra Kumar Choudhary provided an overview of the role of the Bureau of Indian Standards in developing and maintaining India's earthquake engineering standards. He explained the rigorous process through which standards are formulated, reviewed, and revised, ensuring that they remain relevant to advances in engineering knowledge and societal needs.

Table 1 Indian Standards on Earthquake Resistant Design

IS No.	Title
1893	Criteria for earthquake resistant design of structures
(Part 1): 2016 (Two Amendments)	General provisions and buildings (sixth revision) (under review)
(Part 2): 2014 (One Amendment)	Liquid retaining tanks (fifth revision)
(Part 3): 2014 (Reaffirmed in 2019)	Bridges and retaining walls
(Part 4): 2024	Industrial structures including stack-like structures (second revision)
(Part 6): 2022	Base isolated buildings
4967: 2025	Earthquake Instrumentation of Dam Projects (first revision)

IS No.	Title
4991 (Part 1): 2024	Criteria for blast resistant design of structures, Part 1: Above Ground Explosions
6922: 1973 (Reaffirmed in 2018)	Criteria for safety and design of structures subject to underground blasts
13827: 1993 (Two Amendments, Reaffirmed in 2018)	Improving earthquake resistance of earthen buildings, Guidelines
13828: 1993 (Three Amendments, Reaffirmed in 2018)	Improving earthquake resistance of low strength masonry buildings, Guidelines
13920: 2016 (Two Amendments, Reaffirmed in 2021)	Ductile design and detailing of reinforced concrete structures subjected to seismic forces, Code of practice (first revision)
13935: 2009 (under revision, Reaffirmed in 2019)	Seismic evaluation, repair and strengthening of masonry buildings, Guidelines (first revision)
15988: 2013 (Reaffirmed in 2018)	Seismic evaluation and strengthening of existing reinforced concrete buildings, Guidelines

A significant focus of his presentation was the ongoing review of **IS 1893 Part 1**, India's principal standard for earthquake-resistant design of structures. While a revised version was released in 2025, subsequent stakeholder feedback and government directives have led BIS to undertake a comprehensive review process expected to continue over the coming months.

Among the major developments under consideration are:

- Introduction of a revised seismic zonation map based on **Probabilistic Seismic Hazard Analysis (PSHA)**, bringing India closer to international seismic hazard assessment practices.
- Extension of design response spectra to periods up to 10 seconds, enabling more accurate analysis of long period and tall structures.
- Enhanced provisions addressing soil-structure interaction, liquefaction assessment, and soil flexibility effects.
- Dedicated requirements for the seismic protection of non-structural components and building services.

The inclusion of non-structural elements marks a particularly important shift. In critical facilities such as hospitals, airports, and emergency response centres, non-structural systems often represent a substantial portion of total investment and functionality. Their failure can severely impair operations even when the primary structural system remains intact.

Advancements in Ductile Design and Seismic Assessment

The webinar also highlighted ongoing revisions to **IS 13920**, India's standard for ductile detailing. The revised framework aims to provide comprehensive guidance across multiple construction typologies, including reinforced concrete, masonry, and steel structures.

Table 2 Revision of IS 13920 and IS 13935

IS No.	Title
IS 13920 (Part 1)	Earthquake resistant design and detailing of structures, Part 1: General Provisions
IS 13920 (Part 5)	Earthquake resistant design and detailing of structures, Part 5: Buildings
IS 13935 (Part 1)	Earthquake Safety Assessment and Retrofitting of Structures, Part 1: General Provisions
IS 13935 (Part 5)	Earthquake Safety Assessment and Retrofitting of Structures, Part 5: Buildings

Similarly, updates to **IS 13935**, covering seismic evaluation and retrofitting of existing structures, introduce a structured multi-level assessment methodology. This progressive approach enables engineers to select evaluation procedures ranging from preliminary code-based assessments to detailed nonlinear analyses depending on project requirements and available information.

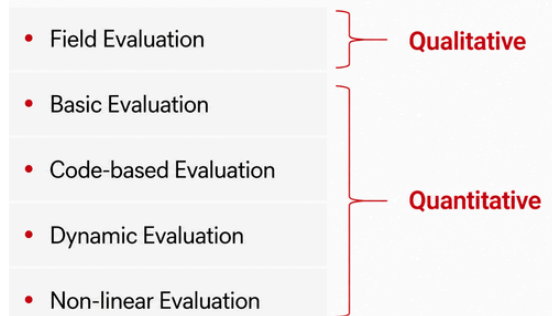


Fig 1 Five-Level telescopic evaluation of structures

In addition to these revisions, BIS is actively developing new standards related to:

- Full-scale structural testing under earthquake loading.
- Seismic energy dissipation devices such as dampers and supplemental damping systems.

These initiatives reflect India's growing emphasis on resilience-based design and performance evaluation.

Global Lessons on Seismic Performance of Connections

Providing an international perspective, Er. Barry Kaknics emphasized that while structural systems receive significant attention during design, the performance of connections frequently determines the actual functionality of a building following an earthquake.

Modern earthquake engineering, he noted, is increasingly moving beyond the traditional objective of preventing collapse. The focus is shifting toward minimizing downtime, reducing economic losses, and ensuring rapid post-earthquake recovery.

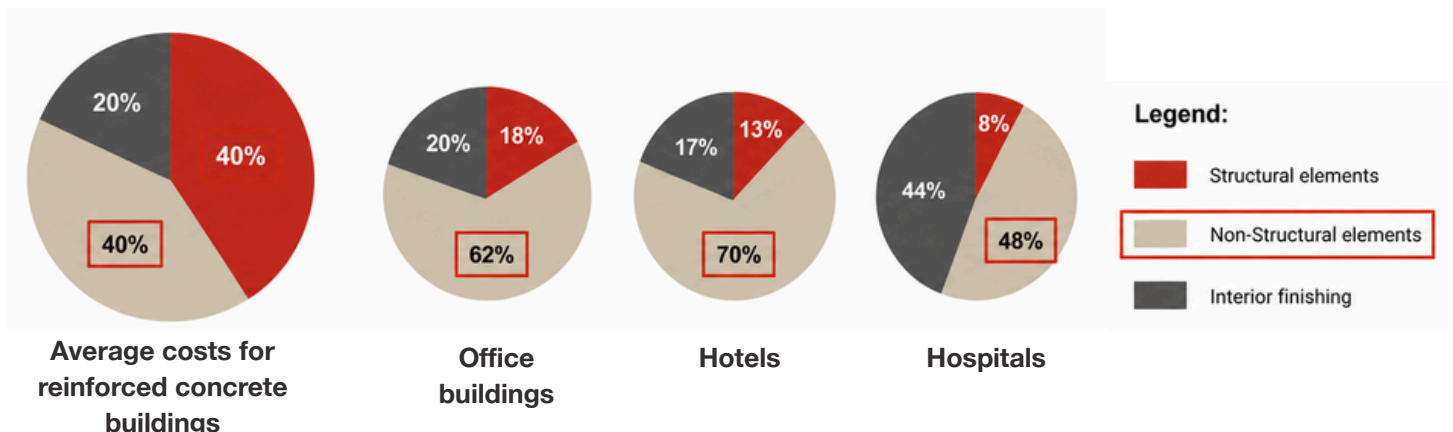


Fig 2 Repair costs after a seismic event

Historical earthquake events offer compelling evidence. During the Christchurch earthquake in New Zealand and the Northridge earthquake in the United States, numerous buildings survived structurally yet became unusable due to failures of ceilings, piping systems, cable trays, facades, and other non-structural components. Several hospitals were forced to suspend operations despite experiencing little or no structural damage.

Seismic Qualification of Post-Installed Anchors

A central theme of the presentation was the seismic performance of post-installed anchors and fastening systems used for both structural and non-structural applications.

The newly published **IS 1946 series** establishes qualification requirements for adhesive and mechanical anchors used in seismic regions. These provisions are based on internationally recognized testing methodologies and represent a major advancement in Indian standards.

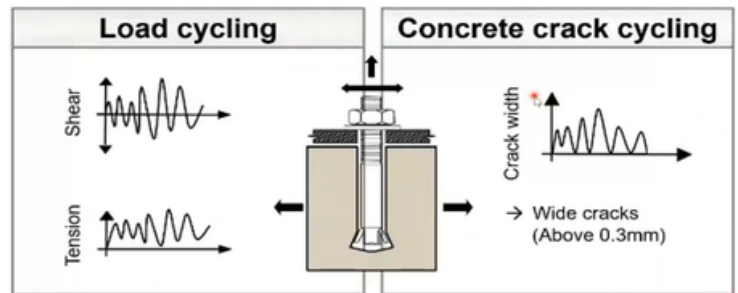


Fig 3 Performance evaluation through load and concrete crack cycling

Notably, India has adopted the more demanding **Seismic Category C2**, which requires anchors to demonstrate reliable performance under severe earthquake conditions. Qualification procedures include:

- Testing in cracked concrete with crack widths up to 0.8 mm.
- Repeated crack opening and closing cycles to simulate seismic load reversals.
- Alternating shear and pulsating tension loading conditions.
- Residual capacity verification following seismic cycling.

Anchors must retain at least 90 percent of their original static capacity after completion of these tests, ensuring that connections remain reliable even after significant seismic demand.

Such stringent qualification requirements support the broader goal of post-earthquake operability and rapid reoccupation of buildings.

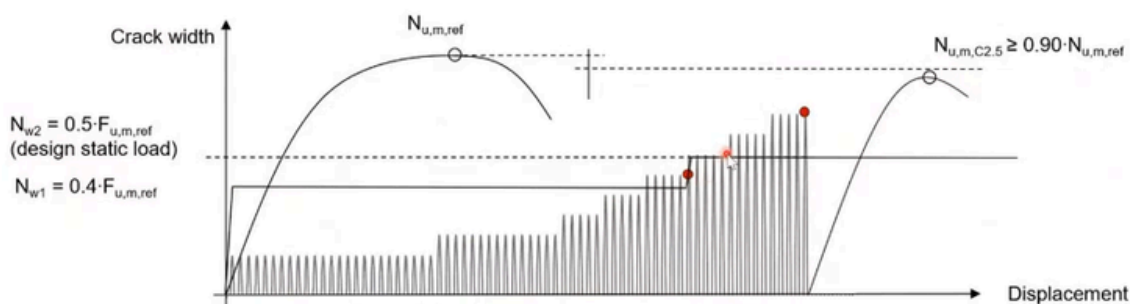
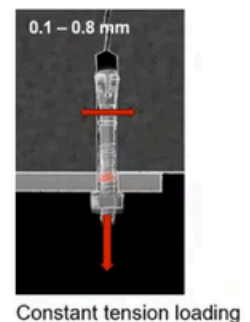


Fig 4 Seismic performance of anchors under cyclic tension loading



Integrating IS 1946 with Seismic Design Practice

During the concluding technical session, Ms. Agamoni Das discussed the integration of the IS 1946 series with broader seismic design provisions.

She explained that IS 1946 Part 2 explicitly requires seismic design considerations for connections located in structures situated within seismic Zones III, IV, and V. The standard references seismic demands established in IS 1893 and requires the use of appropriately qualified anchors capable of performing under cracked concrete and cyclic loading conditions.

A particularly important development is the proposed inclusion of explicit provisions for architectural elements and building utilities within the revised IS 1893 framework. These provisions establish methodologies for calculating seismic forces acting on non-structural components based on factors such as component mass, elevation within the building, and dynamic amplification characteristics.

Such integration ensures that seismic design is approached as a complete system rather than a collection of isolated structural checks.

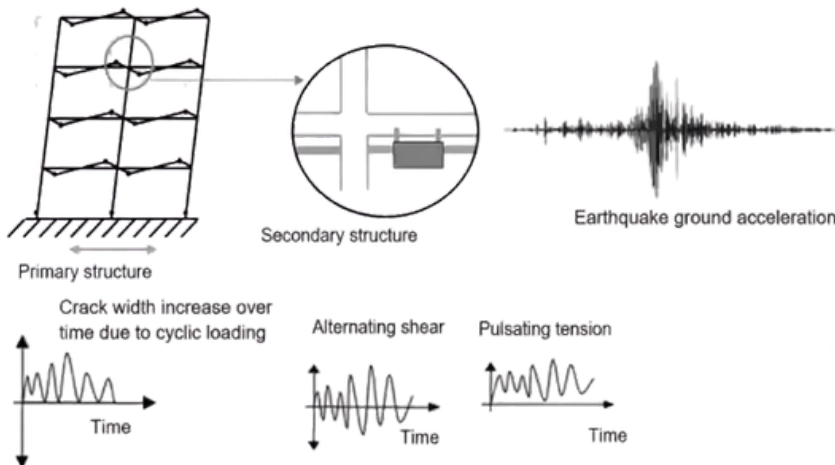


Fig 5 Seismic response of secondary structures under earthquake-induced cyclic loading

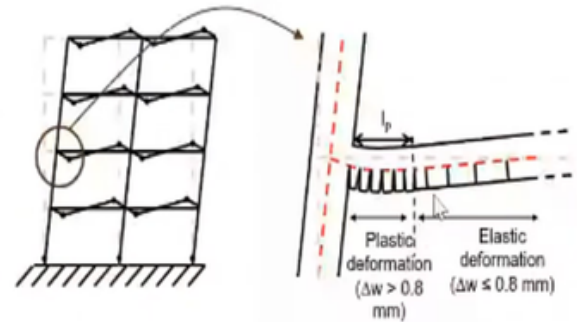


Fig 6 Plastic and elastic portions of reinforced concrete members

Toward a More Resilient Built Environment

The webinar highlighted a broader transformation occurring within the earthquake engineering community. Around the world, resilience is becoming a primary design objective. Engineers are increasingly evaluating buildings not only in terms of life safety but also in terms of functionality, recoverability, and economic continuity.

India's ongoing revisions to seismic standards, combined with the introduction of the IS 1946 anchor qualification framework, represent important steps in this direction. The growing emphasis on non-structural performance, connection design, and performance-based engineering reflects international best practices while addressing the unique challenges of the Indian built environment.

Perhaps the most memorable message of the webinar was the simple yet powerful statement:

“ Seismic survival starts at the connection. ”

As seismic design continues to evolve, ensuring the integrity of both structures and their connections will remain fundamental to protecting lives, safeguarding assets, and enabling communities to recover rapidly after future earthquakes.

Seismic Academy remains committed to facilitating knowledge exchange among researchers, practitioners, code developers, and industry stakeholders. Through such initiatives, the Academy continues to contribute to the advancement of earthquake engineering and the development of a safer, more resilient built environment.



YouTube

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ACADEMY

WEBINAR

May 28th, 2026

Strengthening & Retrofitting of Structures: Post-installed Solutions

The Indian Association of Structural Engineers (IAStructE) organized an insightful technical lecture on “Strengthening & Retrofitting of Structures – Post-installed Solutions” on 28 May 2026. The webinar was delivered by **Ms. Agamoni Das, Head – Codes & Approvals (Fastening Systems), Hilti India**, and attracted participation from structural engineers, academicians, consultants, and students from across the country.

Introducing the speaker, the session chair highlighted Ms. Das’s extensive experience in structural strengthening and seismic mitigation projects. A PhD scholar in Earthquake Engineering at Jadavpur University, Ms. Das has been actively engaged in research on seismic resilience, base isolation, and disaster mitigation techniques. Her professional expertise and research background provided valuable perspectives on the growing importance of strengthening existing infrastructure.



The Pancake Collapse-
Soft Storey Failure



Torsional Irregularity



The Short Column Effect



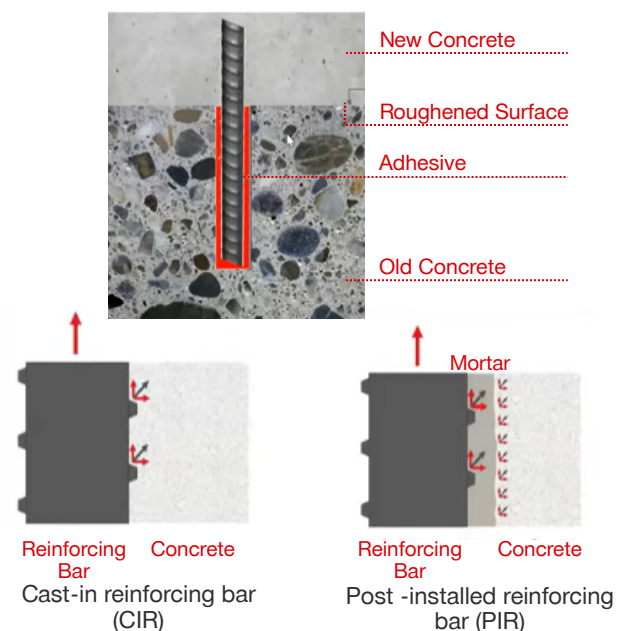
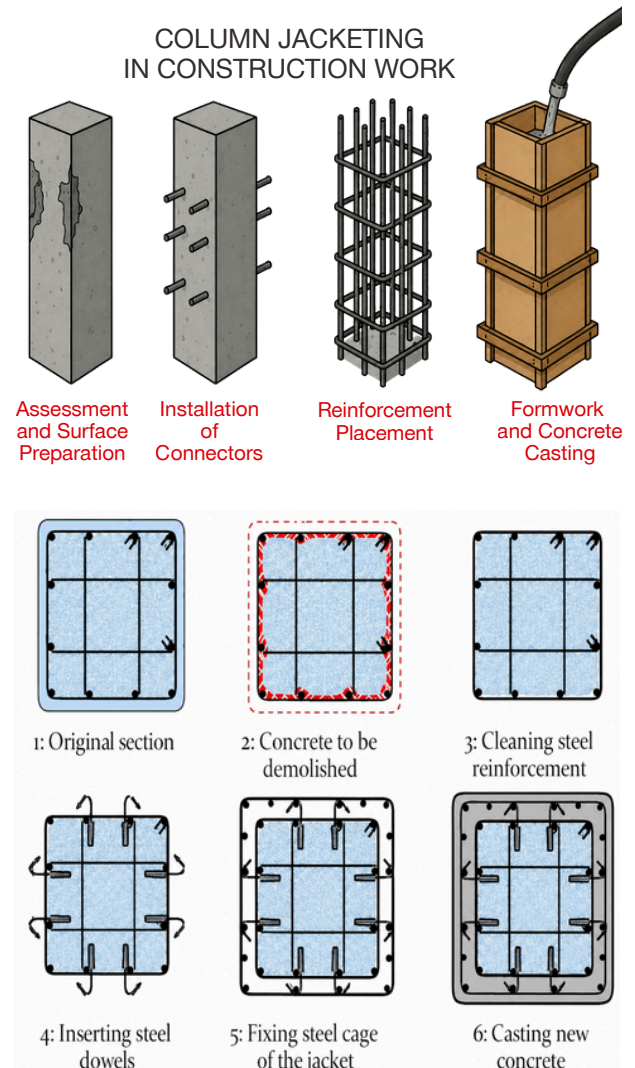
The Soil Liquefaction Lie-
Foundation Failure

The lecture commenced with an overview of the factors responsible for structural deterioration and failure. Ms. Das explained that aging infrastructure, changes in functional requirements, design deficiencies, construction defects, environmental degradation, and natural hazards such as earthquakes and fire often necessitate intervention in existing structures. Drawing examples from past earthquake events, including the Bhuj earthquake, she emphasized the importance of understanding structural vulnerabilities and adopting appropriate retrofitting measures to improve safety and resilience.

A significant portion of the presentation focused on the distinction between **repair**, **rehabilitation**, and **retrofitting**. While repair aims at restoring the original condition of a structure and rehabilitation seeks to regain serviceability, retrofitting involves enhancing the performance and load-carrying capacity of an existing structure beyond its original capabilities. The speaker highlighted that retrofitting has become increasingly relevant in recent years as engineers strive to extend the service life of structures while minimizing demolition waste and reducing environmental impact.

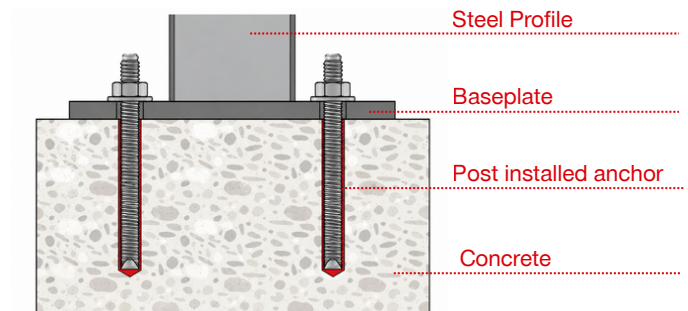
The audience was introduced to various strengthening strategies, broadly classified into global and local retrofitting techniques. Global approaches such as seismic bracing, shear walls, base isolation systems, and energy dissipation devices were discussed as methods for improving overall structural response. At the member level, techniques including concrete jacketing, steel jacketing, fiber-reinforced polymer wrapping, and post-installed reinforcement systems were presented as practical solutions for enhancing the performance of individual structural components.

Emphasis was placed on the role of post-installed solutions, which have emerged as an integral component of modern strengthening technologies. Ms. Das explained how post-installed rebars are used to establish effective load transfer between existing and newly added concrete during concrete jacketing and structural extensions. Similarly, post-installed anchors enable efficient attachment of steel plates and structural elements in steel jacketing applications. The advantages of these systems include flexibility in installation, minimal disruption to existing operations, reduced construction time, and improved sustainability compared with conventional reconstruction approaches.



The lecture also addressed important design considerations associated with post-installed systems. Reference was made to the recently published **IS 1946 (Part 2): 2025**, which provides design provisions for post-installed anchors in concrete.

The speaker discussed the importance of considering static, seismic, fire, and fatigue loading conditions while designing such connections.



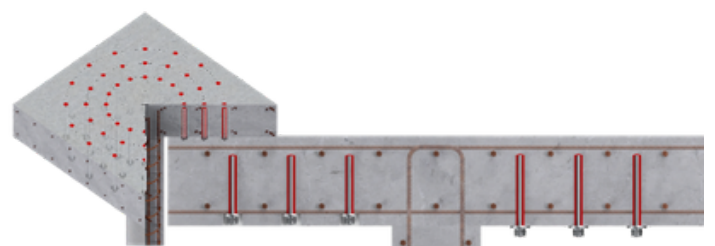
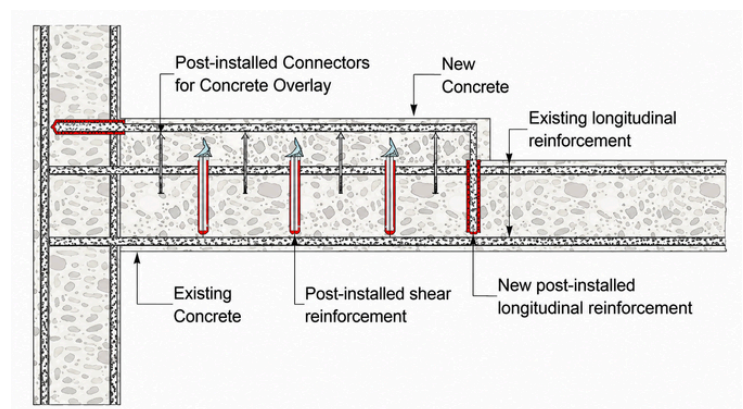
Special attention was given to fire performance, where the behavior of adhesive mortars becomes critical. Proper selection of mortar systems, adequate embedment depths, and sufficient concrete cover were identified as key requirements for ensuring reliable performance under elevated temperatures.

Another interesting segment of the lecture dealt with **HIT-Shear technology**, a post-installed shear reinforcement system used to enhance shear and punching shear resistance in existing concrete structures. Through practical examples, Ms. Das demonstrated how such systems can strengthen beam and slab-column connections and structural members without significantly increasing dead load. Several international case studies were presented to illustrate the successful application of these technologies in strengthening buildings and infrastructure projects.

The presentation concluded with a discussion on structural assessment and the importance of condition evaluation before undertaking retrofitting work.

Visual inspections, non-destructive testing methods, and structural analysis were highlighted as essential tools for determining the suitability of strengthening measures and ensuring effective implementation.

Visual inspections, non-destructive testing methods, and structural analysis were highlighted as essential tools for determining the suitability of strengthening measures and ensuring effective implementation.



In his closing remarks, the session chair, **Dr. S. K. Dhawan**, appreciated the speaker for delivering a technically rich and practically relevant presentation. He noted that strengthening and retrofitting technologies are becoming increasingly important as engineers seek sustainable approaches to maintain and upgrade aging infrastructure. The session concluded with a vote of thanks to the speaker and participants for their active engagement.

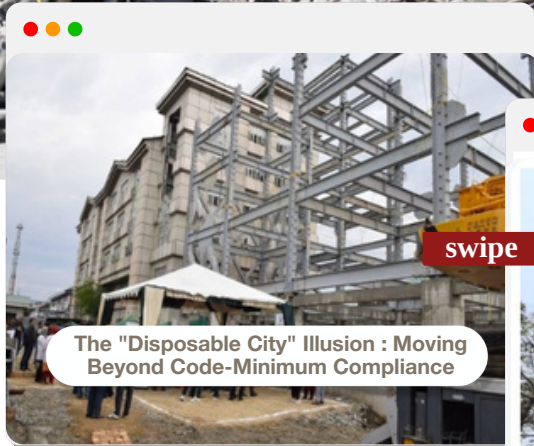
The webinar provided valuable insights into contemporary retrofitting practices and highlighted the growing role of post-installed solutions in improving the safety, durability, and resilience of existing structures.



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When Overhead Reservoirs Fail: The Forgotten Seismic Risk Above Our Cities

A Partially Collapsed Apartment Building in the Marina District

Source : Galloway, K. (2025, October 17). Worry over USGS staffing as Bay Area marks 36 years since Loma Prieta earthquake. KTVU FOX 2 San Francisco.
<https://www.ktvu.com/news/loma-prieta-earthquake-36-years>



Seismic Chronicles:

San Francisco City Hall and the Art of Seismic Retrofitting Loma Prieta Earthquake, 1989

On the evening of **17 October 1989**, the San Francisco Bay Area was shaken by one of California's most significant earthquakes in recent history. With a **magnitude of 6.9**, the Loma Prieta earthquake originated in the **Santa Cruz Mountains** along the mighty **San Andreas Fault**, where the Pacific and North American tectonic plates continually grind past one another.

In a matter of seconds, bridges buckled, freeways collapsed, and entire sections of infrastructure were left in ruins. The earthquake claimed 63 lives, injured more than 3,700 people, and caused nearly \$6 billion in damage. Images of collapsed sections of the San Francisco-Oakland Bay Bridge and the Cypress Street Viaduct came to symbolise the scale of devastation and exposed the vulnerability of the region's design and architecture.

Amid the widespread destruction, another question emerged: How do we protect the buildings that define a city's identity?

One such structure was the San Francisco City Hall.

A Civic Landmark at Risk

Completed in **1915**, San Francisco City Hall is one of the most celebrated examples of **Beaux-Arts architecture** in the United States. With its monumental dome, grand staircases, and richly ornamented

interiors, the building is both a functioning civic centre and an architectural icon. Completed in **1915**, San Francisco City Hall is one of the most celebrated examples of **Beaux-Arts architecture** in the United States. With its monumental dome, grand staircases, and richly ornamented interiors, the building is both a functioning civic centre and an architectural icon.

Yet, like many historic masonry buildings of its era, City Hall had been designed long before modern seismic codes existed. The Loma Prieta earthquake exposed its vulnerabilities. Although the building remained standing, it suffered damage that highlighted the need for a comprehensive **seismic retrofit**.

The challenge was clear: how could the building be strengthened without compromising its historic character?

The Retrofit Solution: Letting the Building Float

After suffering extensive damage during the Loma Prieta Earthquake, the structure required necessary repairs and a seismic retrofit to restore the building's stability



The San Francisco City Hall

Source: San Francisco City Hall, Seismic Retrofit | Forell/Elsesser Engineers, Inc. (n.d.). <https://forell.com/projects/san-francisco-city-hall-seismic-retrofit>

and architectural beauty.

Rather than making the structure heavier or more rigid, engineers adopted a revolutionary approach known as **base isolation**.

The principle is deceptively simple. Instead of forcing a building to resist every movement of the ground, base isolation allows it to **move independently from the earthquake beneath it**.

To achieve this, the entire structure was carefully separated from its foundations and placed on **530 base isolators**, large bearings made of alternating layers of steel and rubber.

During an earthquake:

- The ground moves.
- The isolators deform and absorb energy.
- The building above experiences significantly reduced shaking.

In essence, during the earthquake, the building floats **above the seismic motion**, reducing structural damage.

The retrofit also introduced:

- **Concrete shear walls** around the light courts to improve lateral stiffness.
- **Steel bracing** at the dome and drum level to strengthen the upper portions of the structure.
- **Extensive repairs** and reinforcement while preserving the building's historic fabric.

A New Foundation for an Old Landmark

Retrofitting San Francisco City Hall was an exercise in balancing heritage conservation and seismic resilience. The intervention required meticulous planning and sequencing.

Construction had to proceed without damaging the ornate interiors, decorative finishes, or monumental spaces that make the building unique. At every stage, engineers ensured that the structure remained stable, even if another earthquake occurred during construction.

The project demonstrated an important lesson: Earthquake Resilience does not always require rebuilding. Sometimes, it requires reimagining how existing buildings perform.

Why Seismic Retrofitting Matters

Many buildings around the world were constructed before modern seismic design standards came into force. Seismic retrofitting strengthens these vulnerable structures, enhancing public safety, protecting communities, and improving long-term resilience.

Common seismic retrofitting strategies include:

- Base isolation systems
- Steel bracing
- Concrete shear walls
- Column and beam jacketing
- Foundation strengthening
- Connection reinforcement

The goal is not to make buildings indestructible, but to protect lives, minimise damage, and ensure buildings remain functional after an earthquake.

Seismic Vulnerability of Masonry Housing in India: Evidence, Research Needs, and Policy Implications

By Pravin Kumar Venkat Rao Padalu

Assistant Professor, Earthquake Engineering Research Centre (EERC),
International Institute of Information Technology Hyderabad (IIIT-H),
Telangana, India



Abstract

Masonry construction constitutes the largest portion of India's housing stock and is one of the most seismically vulnerable building typologies in the country. Recurrent earthquake reconnaissance and systematic research document seismic weaknesses in materials, details and connections that produce life-threatening collapse of unreinforced and weakly detailed masonry. To reduce national seismic risk, it is essential to combine rigorous experimental, analytical and numerical research at material, component and structural scales with a coherent policy framework that mandates systematic assessment and large-scale retrofitting of existing masonry housing. This note summarizes the seismic context, vulnerability mechanisms, assessment, and retrofit priorities, and emphasizes the need for sustained, multiscale research to inform policy formulation, code development, and engineering practice.

1. Introduction

India's tectonic setting subjects large populations to seismic hazard arising from the Himalayan collision system, regional intraplate sources, and the Andaman-Nicobar subduction margin; earthquakes in western, northern and northeastern India over recent decades have repeatedly exposed the fragility of the masonry building stock. The predominant dwelling typologies, unreinforced clay-brick, stone and block masonry with mud or weak cement mortar, remain widespread in rural and small-town areas and account for the single greatest share of the national housing inventory. Recent damaging events, including the 2001 Bhuj and 2015 Nepal earthquakes, illustrate the scale of masonry losses in these regions. Because a substantial portion of this inventory was built without engineering oversight or prior to modern seismic provisions, the aggregate exposure of people and assets is exceptionally high and represents a fundamental driver of potential earthquake losses.

2. Seismic Vulnerability and Observed Performance

Masonry is intrinsically brittle, with near-zero tensile capacity and limited ductility; Indian masonry assemblies frequently suffer from low mortar strength, irregular unit geometry and poor workmanship, which materially reduce both in-plane shear resistance and out-of-plane flexural capacity. In addition, significant variability in construction quality, workmanship, and informal building practices across regions further amplifies seismic vulnerability, yet remains inadequately quantified in experimental studies and analytical models of Indian masonry structures. Roof systems that are heavy or poorly anchored and the common absence of continuous horizontal confinement exacerbate seismic demand and promote brittle modes of failure.

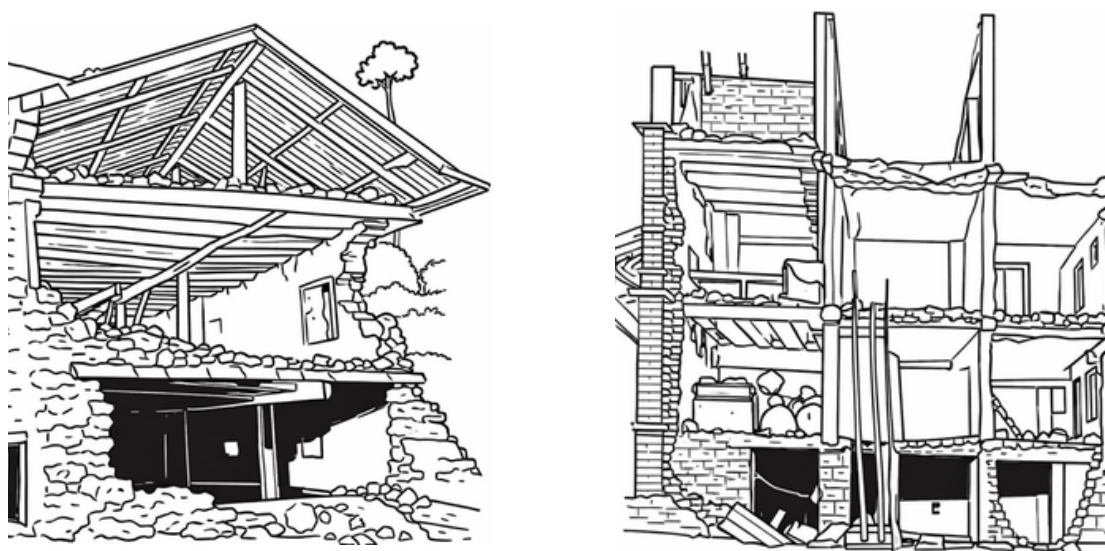


Fig 1 Out-of-plane failure observed in stone- and brick-masonry buildings

Reconnaissance studies and experimental investigations have repeatedly observed the same suite of mechanisms, including out-of-plane wall overturning (refer to Fig. 1), in-plane diagonal shear cracking and bed-joint sliding, separation of wall intersections, and progressive collapse initiated by parapet or roof failure across diverse Indian earthquakes, confirming that poor performance is systematic rather than isolated. These empirical findings are supported by analytical vulnerability studies that show low capacity and unfavourable capacity-demand characteristics for representative Indian URM (unreinforced masonry) typologies.

3. Assessment and the Imperative for Multiscale Research

Effective assessment and prioritisation of interventions require validated material data and modelling tools calibrated to Indian masonry types. Rapid visual screening is a practical first step to identify high-risk units for detailed study, but screening must be complemented by laboratory characterization of the unit and mortar properties, component-level tests on walls and connections under combined in-plane and out-of-plane demands, and full-scale or sub-assembly experiments to capture global response and progressive collapse mechanisms. Numerical approaches, ranging from calibrated macro-element models to refined finite-element simulations, are indispensable for translating experimental results into predictive tools, fragility curves and large-scale vulnerability mapping. In addition, there is a significant lack of reliable seismic fragility and loss models for Indian masonry housing, which limits the translation of structural response into expected damage, economic loss, and life-safety risk at regional and national scales. Development of empirically and analytically calibrated fragility functions is therefore essential for risk-informed planning and prioritization of retrofit interventions. The literature demonstrates both the feasibility and the necessity of such integrated research programmes; without experimental calibration and numerical validation, assessment protocols and retrofit recommendations remain uncertain and difficult to standardize.

Furthermore, the effects of ageing, material deterioration, environmental exposure, and cumulative damage from past seismic events on the seismic performance of masonry buildings remain poorly understood and are seldom incorporated into current assessment and modelling frameworks. Addressing these limitations, therefore, requires sustained and coordinated efforts from the research community, encompassing long-term experimental programmes, development of calibrated analytical models, and validation of numerical tools tailored to Indian masonry typologies. Continuous engagement between researchers, code-writing bodies, and policymakers is essential to translate scientific findings into actionable standards and guidelines. Without such concerted efforts, masonry dwellings, constituting the majority of the national housing stock, will remain inadequately protected against future earthquakes.

4. Retrofitting Strategies and Implementation Requirements

Practical retrofit objectives for existing masonry housing are to restore continuity, provide confinement, reduce overturning propensity, and improve diaphragm-to-wall anchorage. Interventions that strengthen wall-to-roof connections, introduce continuous horizontal bands or confined elements, replace heavy roofs with lighter diaphragms, and provide local reinforcement at openings have demonstrated significant capacity improvements in experimental and numerical studies. Techniques such as ferrocement bands, externally bonded textile or fabric-reinforced cementitious matrix (FRCM) systems, fibre-reinforced polymer (FRP) systems, steel-reinforced grout (SRG), and confined masonry approaches show promise when adapted to local materials and execution skills, provided their effectiveness is demonstrated by localized experimental programmes and life-cycle assessments. For widespread adoption, these technical solutions must be codified into simplified guidelines that account for cost, constructability and cultural constraints, and be paired with incentives and institutional mechanisms for implementation. Equally important is the limited research on scalability, long-term performance, and quality control of retrofitting interventions when implemented at community or regional scales, which poses challenges for large-scale adoption within national seismic risk reduction programmes.

5. Policy Implications and Programmatic Priorities

Mitigating the large-scale seismic risk posed by masonry housing requires a policy framework that integrates research, regulation and implementation. Priority actions include national-scale inventory mapping and screening, funding for targeted experimental and numerical studies to populate fragility databases, development of standardized assessment and retrofit protocols tailored to Indian typologies, financial and risk transfer mechanisms, including insurance incentives, to subsidize retrofit for vulnerable households, and structured capacity building for engineers and masons. Research outputs must be translated into practicable code provisions and retrofit manuals so that interventions are evidence-based and scalable. The literature

highlights the persistent research gaps, especially in region-specific material characterization and validated fragility functions, and demonstrates that policy without research-based technical underpinnings will be insufficient to reduce vulnerability at scale.

A critical limitation in advancing the seismic safety of masonry structures in India is the outdated status of the national masonry design standard. IS 1905, which governs the structural design of masonry, has not undergone a comprehensive revision since 1987 and continues to rely predominantly on working stress concepts. Further, IS 1905 does not include provisions for reinforced masonry construction, a system with recognized potential for improved seismic performance, limiting design guidance for its adoption. This has resulted in a disconnect between contemporary performance-based seismic design philosophies and current masonry design practice. The absence of a limit state-based framework, coupled with limited consideration of seismic demand and deformation capacity, restricts the applicability of the code for both new construction and retrofit design in seismic regions.

Furthermore, the lack of reliable, statistically representative material property data remains a fundamental barrier to both code development and vulnerability assessment. Comprehensive experimental data on compressive strength, shear strength, flexural tensile strength, stiffness degradation, and cyclic degradation characteristics of masonry assemblages representative of Indian construction practices are scarce. This deficiency directly affects analytical and numerical modelling accuracy, fragility estimation, and the formulation of rational design and retrofit provisions. Establishing region-specific material databases through systematic testing is therefore essential for enabling limit state-based design and performance-oriented assessment methodologies.

In addition, there is an urgent need to explicitly incorporate retrofit design approaches based on advanced composite systems, such as FRCM,

FRP and SRG, within IS 13935, which governs the seismic evaluation and strengthening of existing masonry buildings. Despite growing experimental and numerical evidence demonstrating the effectiveness of these systems in enhancing the strength, deformation capacity, and integrity of masonry structures, their absence from the current code framework limits adoption in professional practice. Inclusion of design guidelines and acceptance criteria for such advanced retrofitting interventions would significantly assist practising engineers in rational design and implementation, and facilitate wider, code-compliant application in seismic risk mitigation programmes.

6. Conclusions

Masonry structures constitute the most extensive and most vulnerable segment of India's housing stock with respect to seismic hazard. Recurrent damage patterns observed in earthquakes, combined with analytical and experimental evidence, indicate an urgent need for coordinated action. This action must combine high-quality experimental programmes at material, component and structural scales with validated analytical and numerical modelling, and must be embedded within a robust policy framework that mandates assessment, prioritizes retrofits, provides financial and institutional support, and invests in human capacity building. Only a sustained, research-driven policy and implementation effort can substantially reduce the disproportionate seismic risk posed by India's masonry housing stock. Such an integrated approach provides a realistic pathway toward enhancing life safety, improving the resilience of existing masonry housing, and reducing earthquake-induced losses in future seismic events in India.

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Site-Specific Liquefaction Potential Assessment for Seismic Design Applications



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Abstract

Liquefaction of saturated cohesionless soils during strong ground shaking remains one of the most critical geotechnical hazards affecting the seismic performance of structures and lifeline systems. Reliable evaluation of liquefaction potential requires a systematic estimation of both the seismic demand imposed by the earthquake loading and the resistance offered by the in-situ soil conditions. Continuous developments in seismic research have led to refined procedures for liquefaction assessment, incorporating updated formulations for Cyclic Stress Ratio (CSR), Stress Reduction Coefficient, and normalization of in-situ test data; as proposed by various researchers. This, in turn, provides a structured way of conventional engineering practice. Objective of the current paper is to present a step-by-step procedure to assess site specific liquefaction potential of soil deposits; based on the simplified stress-based approach, synthesized from established procedures reported in the literatures. Emphasis is placed on the evaluation of Cyclic Stress Ratio induced by earthquake loading, estimation of Cyclic Resistance Ratio (CRR) from Standard Penetration Test (SPT) data of cohesionless soil; and from Over-Consolidation Stress Ratio (OCR) of cohesive soil. Further, appropriate correction factors are applied, and Factor of Safety against liquefaction for individual soil layers are computed. The suggested methodology further incorporates post-liquefaction volumetric strain assessment to estimate potential ground settlement. Thus, this method provides a comprehensive framework for practical engineering application.

1. Introduction

Soil liquefaction may be defined as a seismic phenomenon in which saturated, loose to medium-dense cohesion less soils experience a significant reduction in their shear strength and stiffness due to excess pore water pressure during earthquake events. The occurrence of liquefaction has been widely experienced in past earthquakes, where it has led to significantly large ground settlements, lateral spreading, sudden loss of bearing capacity, and moderate to severe damages of civil engineering infrastructures. Consequently, the evaluation of liquefaction potential seems to be an essential component of geotechnical design in seismic active regions [2]. In this context, the significance of such studies is further emphasized by the widespread seismic vulnerability observed across large parts of the Indian subcontinent.

To facilitate practical engineering assessment, simplified stress-based procedures have been widely adopted for estimating the susceptibility of soil deposits towards liquefaction [1, 3, 7]. These methods compare the Cyclic Shear Stress induced by the seismic excitations with Cyclic Resistance of the soil, often derived from in-situ test data like: Standard Penetration Test (SPT) or Cone Penetration Test (CPT) for granular soil; and Over-Consolidation Stress Ratio (OCR) for fine-grained soils.

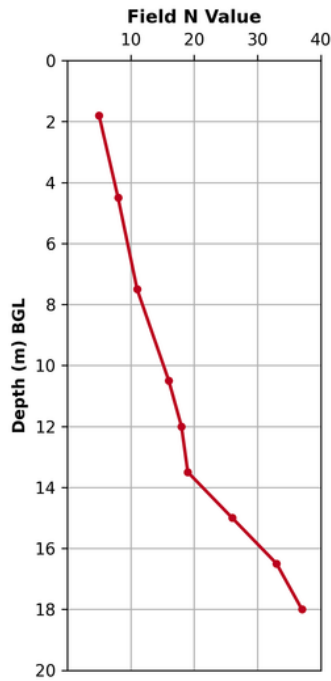
While simplified in nature, the reliability of these procedures depends mainly on appropriate correction of field test results, accurate estimation of seismic demand parameters, and adherence to the applicability limits prescribed in different literatures.

This study aims to systematically demonstrate the liquefaction assessment procedure through a step-by-step evaluation approach [1,3]. SPT-based liquefaction analysis is shown here for sandy soil strata, including correction of raw blow counts, estimation of cyclic stress and resistance ratios, determination of factor of safety against liquefaction, and assessment of post-liquefaction volumetric strain. For clayey strata, OCR-based evaluation approach is employed [7]. A structured framework for evaluating Cyclic Stress Ratio, Cyclic Resistance Ratio, and Factor of Safety against liquefaction is used [3, 5-7] in the current paper. The formulations for stress reduction coefficient, magnitude scaling effects, and normalization of in-situ test results; reflecting the actual seismic hazard characterization for a specific site are taken from [3]. The proposed framework is intended to provide a transparent and practical reference for engineers and researchers engaged in seismic geotechnical analysis.

2. Description of the Problem Statement

Table 1 SPT values and description of soil strata

Depth (m)BGL		Type of Sample	SPT record				Strata Description			
From	To		0–150	150–300	300–450	N	Depth (m)	Thick ness (m)	Description	Strat um
1.5	1.95	P	2	2	3	5	0–5	5	Medium stiff brownish-grey silty clay with kankars. (CH)	I
3	3.45	U	NA							
4.5	4.95	P	2	3	5	8				
6	6.45	U	NA				5–11	6	Stiff mottled yellow yellow silty clay with calcareous nodules, kankars & rusty spots. (CH)	II
7.5	7.95	P	3	4	7	11				
9	9.45	U	NA							
10.5	10.95	P	5	7	9	16				
12	12.45	P	6	8	10	18	11–18	7	Medium dense/ dense yellow silty sand with traces of mica. (SM-SP)	III
13.5	13.95	P	7	9	10	19				
15	15.45	P	9	12	14	26				
16.5	16.95	P	11	15	18	33				
18	18.45	P	12	17	20	37				



The measured SPT values and corresponding stratigraphic details, based on bore log reports and sieve analysis data for a site located in a seismically active region, are presented in Tables 1 and 2. The variation of field SPT N-values with depth is illustrated along with. The groundwater table is located at a depth of 1.5 m below ground level. Based on these data, liquefaction potential of the soil profile and the associated post-liquefaction ground settlement are evaluated. The assessment is carried out considering the proposed construction of an important reinforced concrete (RCC) building.

3. Evaluation of Liquefaction Potential

The detailed step-by-step calculation of factor of safety against liquefaction is shown for a typical set of parameters corresponding to the cohesion less soil sample taken at 12.00 m BGL. In case of clayey strata samples, a typical sample taken at 3.00 m BGL is considered for further detailed calculations. Tabulated reports for all the samples are given in Table 5

Table 2 Grain size distribution

Stratum	From (m)	To (m)	Gravel	Sand	Silt	Clay	Percentage Fine. (%)
I	1.5	1.95	0	3	65	32	97
	4.5	4.95	0	2	62	36	98
II	7.5	7.95	2	2	56	40	96
	10.5	10.95	1	3	58	38	96
III	12	12.45	0	89	11	0	11
	13.5	13.95	0	88	12	0	12
	15	15.45	0	87	13	0	13
	16.5	16.95	0	89	10	1	11
	18	18.45	0	88	10	2	12

3.1 Estimation of Cyclic Stress Ratio

The Cyclic Stress Ratio (CSR) is evaluated following the procedure reported in [3]. The seismic demand on the soil is estimated using the following expression:

$$CSR = 0.65 \left(\frac{a_{Max}}{g} \right) \left(\frac{\sigma_{vo}}{\sigma'_{vo}} \right) r_d$$

Where, a_{max} = Peak Ground Acceleration (PGA); σ_{vo} = total vertical overburden stress at the chosen depth in the potentially liquefiable layers within the deposit; r_d = stress reduction coefficient $e^{\alpha_z + \beta_z M}$; g = acceleration due to gravity; and σ'_{vo} = effective vertical overburden stress at the chosen depth in the potentially liquefiable layers within the deposit. M = magnitude of the largest likely earthquake, assumed as 7.5 based on regional seismicity considerations.

In absence of site-specific peak ground acceleration (PGA) data, the ratio (a_{Max}/g) is assumed to be 0.5 for present analysis, representing conservative estimate of seismic demand.

Hence, $\left(\frac{a_{Max}}{g}\right) = 0.5$ Now, α_z, β_z, r_d and CSR are

calculated below.

$$\begin{aligned}\alpha_z &= -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) \\ &= -1.012 - 1.126 \sin\left(\frac{12}{11.73} + 5.133\right) = -0.869\end{aligned}$$

$$\begin{aligned}\beta_z &= 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right) \\ &= 0.106 + 0.118 \sin\left(\frac{12}{11.28} + 5.142\right) = 0.097\end{aligned}$$

$$\begin{aligned}r_d &= e^{\alpha_z + \beta_z M} \\ &= e^{-0.869 + 0.097 \times 7.5} = 0.867\end{aligned}$$

$$\begin{aligned}CSR &= 0.65 \left(\frac{a_{Max}}{g}\right) \left(\frac{\sigma_{vo}}{\sigma'_v}\right) r_d \\ &= 0.65 \times 0.5 \times \left(\frac{220.4}{102.8}\right) \times 0.867 = 0.604\end{aligned}$$

3.2 Estimation of Cyclic Resistance Ratio (CRR)

This section outlines the methodology adopted [3, 5-7] for the evaluation of Cyclic Resistance Ratio (CRR) for both sandy and clayey soil strata.

3.2.1 Estimation of CRR for cohesion less strata

The Cyclic Resistance Ratio (CRR) is evaluated following the procedure reported in [3]; based on the SPT Data of cohesion less strata. The capacity of the soil is estimated using the following expression:

$$CRR = CRR_{M=7.5, \sigma'_v=1 \text{ atm.}} (MSF) K_\sigma [6]$$

Where, MSF= magnitude scaling factor =

$$1 + (MSF_{Max} - 1)(8.64e^{-0.25M} - 1.325) [6]$$

K_σ = overburden stress correction factor =

$$\left[1 - C_\sigma \ln\left(\frac{\sigma'_{vo}}{P_a}\right)\right] \leq 1.1 [5]$$

In which, MSF_{Max} = maximum value of magnitude scaling factor = $1.09 + \left(\frac{(N_1)_{60cs}}{31.5}\right)^2 \leq 2.2$ from SPT [6]

$$\text{And, } C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60cs}}} \leq 0.3, [5] \text{ from SPT,}$$

M= magnitude of largest likely earthquake,

P_a = atmospheric pressure. Here, M=7.5 is assumed for the specific site.

Now, the $(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$; where,

$$(N_1)_{60} = C_N N_{60}; \text{ and } N_{60} = C_E C_R C_B C_S N_m.$$

in which

$$\Delta(N_1)_{60} = e^{1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2};$$

$$C_N = \left(\frac{P_a}{\sigma'_v}\right)^m \leq 1.7,$$

wherein $m = 0.784 - 0.0768 \sqrt{(N_1)_{60cs}} [3, 5-7]$

For the selected sample (i.e., @ 12.00 m depth BGL), $N_m=18$. As the SPT is conducted as per IS 2131, the energy delivered to the drill rod is about 60 percent. Thus, C_E may be assumed as 1. As per the site-specific sub-soil exploration testing equipment, the Borehole diameter factor, $C_B=1$, Rod length factor, $C_R=1$, Sampler factor, $C_S=1$. Therefore,

$$N_{60} = C_E C_R C_B C_S N_m = 1 \times 1 \times 1 \times 1 \times 18 = 18.$$

The corresponding $C_N = 0.988$ is evaluated using iterations. Hence,

$$(N_1)_{60} = 0.988 N_{60} = 0.988 \times 18 = 17.779.$$

Now,

$$\begin{aligned}\Delta(N_1)_{60} &= e^{1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2} \\ &= e^{1.63 + \frac{9.7}{11+0.01} - \left(\frac{15.7}{11+0.01}\right)^2} = 1.6122\end{aligned}$$

This yields the value of

$$(N_1)_{60cs} = 17.779 + 1.6122 = 19.391$$

The equation $m = 0.784 - 0.0768 \sqrt{(N_1)_{60cs}}$ provides the value of, $m = 0.446$. So,

$$C_N = \left(\frac{P_a}{\sigma'_v}\right)^m = \left(\frac{100}{220.4}\right)^{0.446} = 0.988 \leq 1.7.$$

Now, for the SPT based calculations,

$$\begin{aligned}CRR_{M=7.5, \sigma'_v=1 \text{ atm.}} &= e^{-2.8 + \left(\frac{(N_1)_{60cs}}{14.1}\right) + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4} \\ &= e^{-2.8 + \left(\frac{19.391}{14.1}\right) + \left(\frac{19.391}{126}\right)^2 - \left(\frac{19.391}{23.6}\right)^3 + \left(\frac{19.391}{25.4}\right)^4} \\ &= 0.19867\end{aligned}$$

$$\text{Now, } MSF_{Max} = 1.09 + \left(\frac{(N_1)_{60cs}}{31.5}\right)^2$$

$$= 1.09 + \left(\frac{19.391}{31.5}\right)^2 = 1.469 \leq 2.2$$

Hence, MSF = Magnitude Scaling Factor

$$= 1 + (MSF_{Max} - 1)(8.64e^{-0.25M} - 1.325)$$

$$1 + (1.469 - 1)(8.64 \times e^{-0.25 \times 7.5} - 1.325) \approx 1.000$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60cs}}} = \frac{1}{18.9 - 2.55\sqrt{19.391}} = 0.130 \leq 0.3,$$

$$K_\sigma = \left[1 - C_\sigma \ln \left(\frac{\sigma'_{vo}}{P_n} \right) \right] = \left[1 - 0.130 \times \ln \left(\frac{102.8}{100} \right) \right] = 0.9964 \leq 1.1$$

$$\begin{aligned} \text{Therefore, } CRR &= CRR_{M_{\pi}=7.5, \sigma'_v=1 \text{ atm.}} (MSF) K_\sigma \\ &= CRR_{M_{\pi}=7.5, \sigma'_v=1 \text{ atm.}} (MSF) K_\sigma \\ &= 0.19867 \times 1.000 \times 0.9964 = 0.198 \end{aligned}$$

Finally, the Factor of Safety

$$FS_{liq} = \frac{CRR}{CSR} = \frac{0.198}{0.604} = 0.328 < 1.1.$$

Hence, this soil layer is unsafe against liquefaction [8]

3.2.2 Estimation of CRR for cohesive strata

For clay and plastic silt, the Cyclic Resistance Ratio (CRR) is evaluated as a function of the over-consolidation ratio (OCR) of the soil within cohesive strata. The corresponding cyclic resistance is estimated using the following expression:

$$CRR = 0.18 (OCR)^{0.8}, \text{ from Empirical Method [7].}$$

Over-Consolidation Stress Ratio (OCR) is the ratio of a soil's maximum past effective vertical stress (Pre-Consolidation Pressure) to its current effective vertical stress; used to define soil stress history and compressibility. The Pre-Consolidation Pressure of three (03) nos. Undisturbed Soil Samples (UDS) taken at different depths for strata I & II are calculated (ref. Fig. 01) using Casagrande's graphical technique, as suggested in IS 8009 (Part-I): 1976 (Re-affirmed 2023) [4].

The Pre-Consolidation Pressure (P_c) at 3.00 m depth BGL is calculated to be 2.1 kg/cm².

$$\text{Therefore, Over-Consolidation Stress Ratio (OCR)} = \left(\frac{P_c}{\sigma'_{vo}} \right) = \left(\frac{2.1 \times 98.1}{25.2} \right) = 8.175$$

$$\text{Hence, } CRR = 0.18 (OCR)^{0.8} = 0.18 (8.175)^{0.8} = 0.967$$

The Factor of Safety

$$FS_{liq} = \frac{CRR}{CSR} = \frac{0.967}{0.691} = 1.3989 \in (1.1, 1.4).$$

Hence, the ground settlement of this soil layer is to be verified based on Fig. 10 of [9]

4. Results & Discussions

The variation of Cyclic Stress Ratio (CSR) along the borehole depth is evaluated, and the corresponding results are presented in Table 3. The decreasing trend of CSR (ref. Fig. 02) suggests about less demand on the subsoil with increasing depth.

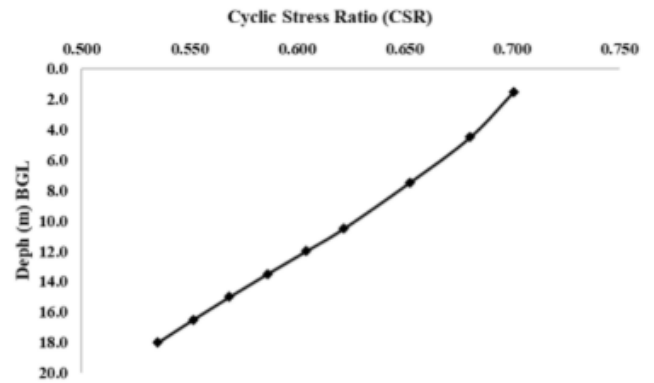


Fig 2 Variation of Cyclic Stress Ratio (CSR) with Depth (m)

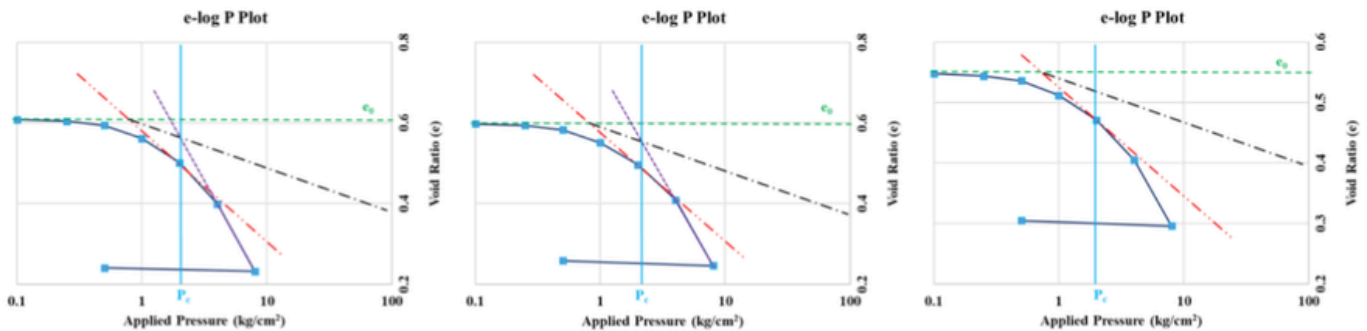


Fig 1 Pre-Consolidation Pressure (P_c) Calculation at (a) 3.0 m, (b) 6.0 m, and (c) 9.0 m depth BGL

Table 3 Cyclic Stress Ratio (CSR) for different soil strata

Stratum	Depth (m) BGL	Field N	Percentage Fine (%)	α_z	β_z	M	r_d	γ_b (kN/m ³)	γ_w (kN/m ³)	σ_{vo} (kN/m ²)	σ'_{vo} (kN/m ²)	$\sigma_{vo} / \sigma'_{vo}$	a_{max}/g	CSR
I	1.5	5	97	-0.051	0.006	7.5	0.995	18.2	9.8	27.3	12.6	2.167	0.5	0.701
	4.5	8	98	-0.231	0.026	7.5	0.966	18.2	9.8	81.9	37.8	2.167	0.5	0.681
II	7.5	11	96	-0.462	0.052	7.5	0.93	18.4	9.8	137	63.5	2.157	0.5	0.652
	10.5	16	96	-0.728	0.081	7.5	0.889	18.4	9.8	192.2	89.3	2.152	0.5	0.622
III	12	18	11	-0.869	0.097	7.5	0.867	19	9.8	220.4	102.8	2.144	0.5	0.604
	13.5	19	12	-1.013	0.113	7.5	0.845	19	9.8	248.9	116.6	2.135	0.5	0.586
	15	26	13	-1.156	0.128	7.5	0.822	19	9.8	277.4	130.4	2.127	0.5	0.569
	16.5	33	11	-1.298	0.143	7.5	0.8	19	9.8	305.9	144.2	2.121	0.5	0.552
	18	37	12	-1.434	0.158	7.5	0.778	19	9.8	334.4	158	2.116	0.5	0.535

The Cyclic Resistance Ratio (CRR) values along the borehole depth are computed and presented in Tables 4 & 5. A plot is given below in Fig. 03 showing the trend of variation of CRR for different soil strata.

Table 4 Cyclic Resistance Ratio (CRR) for cohesion less soil stratum

Estimation of Cyclic Resistance Ratio (CRR)																			
Stratum	Depth (m) BGL	Field N	Percentage Fine (%)		C_R	C_b	C_s	N_{60}	C_N	$(N_1)_{60}$	ΔN	$(N_1)_{60}^{CS}$	m	MSF_{max}	MSF	C_σ	K_σ	CRR_{Mw}	CRR
III	12	18	11	1	1	1	1	18	0.988	17.78	1.612	19.39	0.446	1.469	1	0.13	0.996	0.1987	0.198
	13.5	19	12	1	1	1	1	19	0.933	17.73	2.073	19.8	0.442	1.485	1	0.132	0.98	0.2035	0.199
	15	26	13	1	1	1	1	26	0.901	23.42	2.508	25.93	0.393	1.768	1	0.169	0.955	0.3138	0.3
	16.5	33	11	1	1	1	1	33	0.876	28.91	2.905	31.81	0.351	2.11	1	0.221	0.919	0.6259	0.575
	18	37	12	1	1	1	1	37	0.855	31.65	2.073	33.72	0.338	2.2	1	0.244	0.888	0.863	0.767

Table 5 Cyclic Resistance Ratio (CRR) for cohesive soil strata

Stratum	Depth (m) BGL	P_c (kN/m ²)	σ'_{vo} (kN/m ²)	OCR	CRR
I	3	206	25.2	8.175	0.967
II	6	211	50.6	4.168	0.564
	9	194	51.6	3.764	0.52

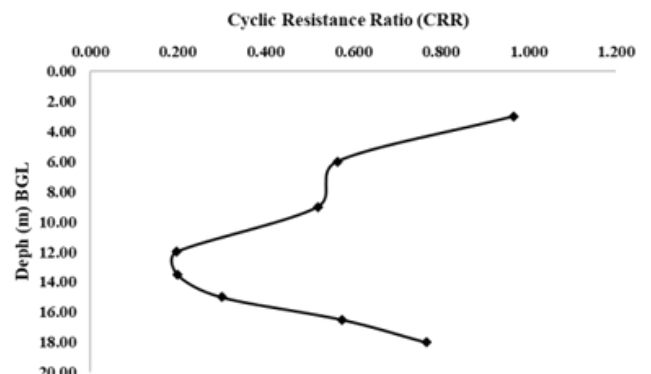


Fig 03 Variation of Cyclic Resistance Ratio (CRR) with Depth (m)

The Cyclic Resistance Ratio (CRR) initially gets decreased along depth for two nos. cohesive strata (I & II). This behaviour is observed mainly due to the increase in effective overburden pressure, but almost uniform pre-consolidation pressure along the sampling depth. For the stratum-III (cohesionless stratum), increase of CRR along borehole depth is mainly associated with the greater Field-N values with higher depth.

Table 6 Factor of Safety against Liquefaction (FS_{liq}) for different soil strata

Stratum	Depth (m) BGL	CRR	CSR	FOS
I	3	0.967	0.691	1.4
II	6	0.564	0.666	0.846
	9	0.52	0.637	0.816
III	12	0.198	0.604	0.328
	13.5	0.199	0.586	0.34
	15	0.3	0.569	0.527
	16.5	0.575	0.552	1.043
	18	0.767	0.535	1.432

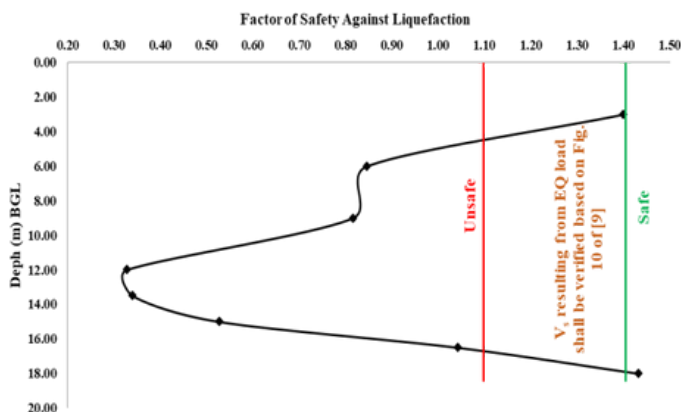


Fig 04 Variation of Factor of Safety against Liquefaction with Depth (m)

The Factor of Safety against liquefaction (FS_{liq}) along the borehole depth is determined and summarized in Table 6. The depth-wise variation of FS_{liq} is depicted in Fig. 04, with demarcation of safe and potentially vulnerable zones.

The FS_{liq} initially gets decreased along depth for the cohesive strata (I & II). This behaviour is observed mainly due to the decrease in OCR, resulting lowered CRR. For the stratum-III (cohesion less stratum), FS_{liq} gets increased with incremental depth; showing better safety against liquefaction.

Factor of safety for the last sample taken at 18.00 m depth BGL is greater than 1.4; thus, safe against liquefaction. Further, ground settlement resulting from earthquake loading is computed based on the method suggested in [9]. Total vertical settlement of stratum-III due to liquefaction is tabulated in Table-6. Depth wise cumulative vertical settlement is presented in Fig. 05.

Although the liquefaction analysis yields FS_{liq} < 1 for stratum-II soil-samples, it is a highly plastic Over-Consolidated clay (OCR ≈ 4). Based on established plasticity criteria, the deposit seems not to be susceptible to classical liquefaction phenomenon. Sand-based correlations are thus not applicable. However, cyclic degradation of undrained strength may be checked for higher seismic demand.

Table 7 Vertical Settlement for the cohesion less soil stratum

Stratum	Depth (m) BGL	CRR	CSR	FOS	ϵ_v (%)	Vertical Settlement, Δ_v (mm)
III	12	0.198	0.604	0.328	2.8	28
	13.5	0.199	0.586	0.34	2.8	42
	15	0.3	0.569	0.527	2.2	33
	16.5	0.575	0.552	1.043	1.7	25.5
	18	0.767	0.535	1.432	0	0
Total Vertical Settlement						128.5

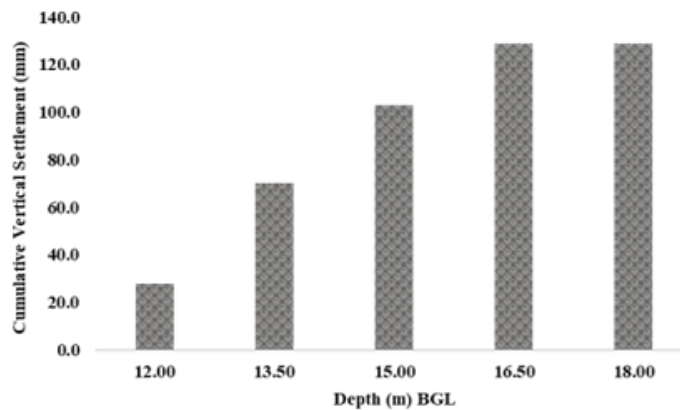


Fig 05 Cumulative Vertical Settlement (mm) vs with Depth Plot

However, detailed index property evaluation indicates that these samples are highly plastic cohesive soils; and are strongly over-consolidated ($OCR \approx 3.8-8.2$). Such soils fall outside the domain of applicability of sand-based liquefaction correlations, which are calibrated for clean sands and low-plasticity silts. For these types of clayey deposits, earthquake response seems to be governed by cyclic softening and limited pore-pressure generation rather than the classical liquefaction (zero effective stress condition). The computed low factors of safety therefore represent a methodological limitation rather than a realistic failure mechanism. This paper seems to have an impact for real life liquefaction assessment of sub-soil profiles.

Acknowledgements

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9. Conclusions

The liquefaction assessment is attempted using SPT-based procedures and OCR-dependent cyclic resistance relationships for the investigated subsoil profile. Some results yielded low factors of safety when evaluated within the conventional CSR-CRR framework. Thus, the ground settlement resulting from seismic excitations is calculated based on the formulae given in different literatures. Some results of a clayey stratum have shown lower factor of safety as per the said formulae.

SEISMIC STRENGTHENING

From Shake to Safe



SEISMIC ACADEMY ANNUAL CONFERENCE

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23RD OCTOBER 2026

FRIDAY

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9:00 AM - 5:00 PM (IST)

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9:00 AM - 10:00 AM	Registration	
10:00 AM - 10:15 AM	Opening Spark	Mr. Jayant Kumar General Manager, Hilti India
10:15 AM - 10:30 AM	5th Anniversary Cake Cutting and SA Retrofitting guidebook Launch, IAStructE Post-installed Rebar guidebook launch	Advisory Board Members, Mr. Alok Bhowmick, President, IAStructE
10:30 AM - 11:00 AM	Keynote Address 1	Prof Sriman Kumar Bhattacharya Vice Chancellor, Shiv Nadar University, Chennai Former Head- Indian Institute of Technology, Kharagpur
11:00 AM - 11:30 AM	Seismic Zonation of India	Prof STG Raghukanth Dept. of Civil Engineering Indian Institute of Technology, Madras
11:30 AM - 12:00 PM	Venezuela Doublet Earthquake of 24 June 2026: Damage to Infrastructure and Way forward for Indian Standards	Dr Ajay Chourasia Chief Scientist Central Building Research Institute, Roorkee
12:00 AM - 12:15 PM	High Tea / Networking	
12:15 PM - 12:45 PM	Global Perspective - Session 1	Dr Andreas Lampropoulos Assistant Professor, School of Civil Engineering National Technical University of Athens (NTUA)
12:45 PM - 1:30 PM	Rising Engineers Competition	Jury Experts (TBC)
1:30 PM - 2:30 PM	Networking lunch	
2:30 PM - 2:55 PM	Industry Insights	TBC
2:55 PM - 3:20 PM	Industry Insights	TBC
3:20 PM - 3:50 PM	Case Study & Challenges	Ms. Sangeetha Vij Managing Partner, S D Engineering Consultants
3:50 PM - 4:30 PM	Interactive Session (Open House)	TBC
4:30 PM - 4:45 PM	Summary & Closing	Mr. Pulkit Kukreja Vice President - Engineering, Hilti India
5:00 PM Onwards	Departures	

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Burj Khalifa

Engineering Seismic Performance at Extreme Height

The Burj Khalifa is the world's tallest structure. For the seismic engineering community, however, its greater significance lies in resolving a fundamental paradox: it is a supertall building in a region of moderate seismic risk whose very form, designed for extreme wind, provides a masterclass in inherent seismic resilience. This article analyses the Burj Khalifa not merely as an architectural icon, but as a definitive case study in integrated multi-hazard design.

It shows how a performance-based philosophy, an aerodynamically derived structure, and a robust geotechnical strategy collectively forge a benchmark for seismic performance at the extreme of height.

Standing at 829.8 meters in Dubai, the Burj Khalifa's design was driven by the immense wind forces at such an altitude. Yet, its engineering solutions, developed to ensure stability and comfort against dynamic wind loads, have resulted in a structural system of exceptional and confirmed seismic robustness. This integrated approach makes it a seminal reference for the design of future supertall structures in regions with complex multi-hazard environmental demands.

The Burj Khalifa was designed for a region classified as **Seismic Zone 2A** under the **1997 Uniform Building Code (UBC-97)**, with a seismic zone factor of **$Z = 0.15$** and founded on **Soil Profile Type Sc** (very dense soil/soft rock). This formal classification defined the base seismic demands. The paramount challenge arose from the structure's dynamic properties: with a fundamental period exceeding 10 seconds, the tower is particularly responsive to **long-period ground motions** capable of being generated by distant regional faults.



To address this, engineers implemented a rigorous **Performance-Based Seismic Design (PBSD)** framework, supplementing code requirements with advanced **nonlinear time-history analyses**. This ensured predictable structural performance, from life-safety to post-earthquake functionality, under specific ground motion scenarios. Crucially, this seismic framework was developed in constructive interaction with an unprecedented wind engineering program involving **over 40 wind tunnel tests**, ensuring that every design decision for wind mitigation concurrently enhanced seismic robustness.

The triple-lobed, **Y-shaped footprint**, inspired by the Hymenocallis flower provides inherent **torsional stability** and symmetrical load distribution. The 27 spiraling setbacks gradually reduce the building's cross-section and mass as it ascends. This tapering is a sophisticated aerodynamic feature that disrupts the formation of coherent wind vortices, a process described by engineers as "confusing the wind".

This geometry provides exceptional **inherent torsional stiffness**, a critical defence against the twisting forces (torsion) induced by uneven ground shaking during an earthquake. The Burj Khalifa's ability to reach its record height while insisting on stability is enabled by its innovative "**buttressed core**" structural system, an evolution of the tubular design ideal for lateral resistance.

System Configuration: At its heart is a massive, **hexagonal reinforced concrete central core**. This core provides the primary resistance to **torsional (twisting) forces**, a critical capability during seismic shaking. It is buttressed by three wing-like structures arranged at 120 degrees, which function as robust shear walls to resist lateral forces. The system is unified and stiffened by several sets of outrigger walls at mechanical floors, which connect the core to perimeter columns, enabling the entire structure to function as a monolithic vertical cantilever.

This configuration creates **multiple, redundant load paths**. This **redundancy** is a core seismic

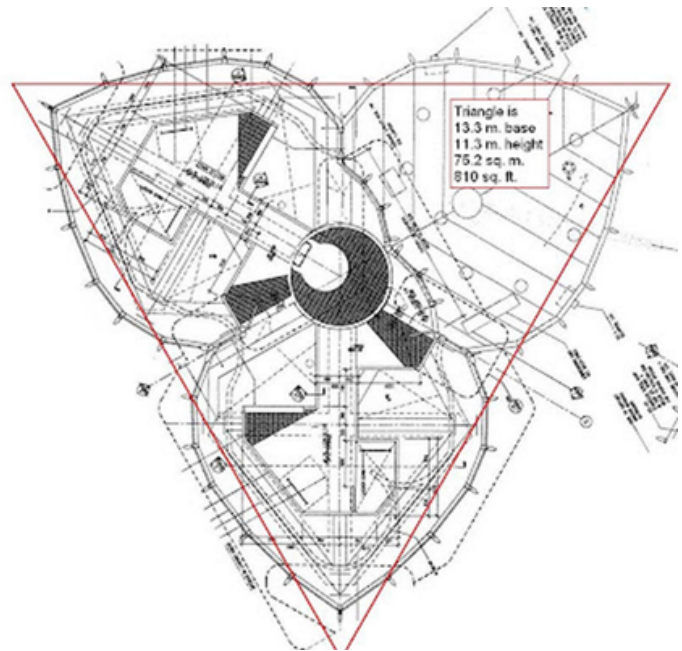


Fig 1 Foundation Design for the Burj Dubai, <https://www.scribd.com/document/770926944/FOUNDATION-DESIGN-FOR-THE-BURJ-DUBAI>.

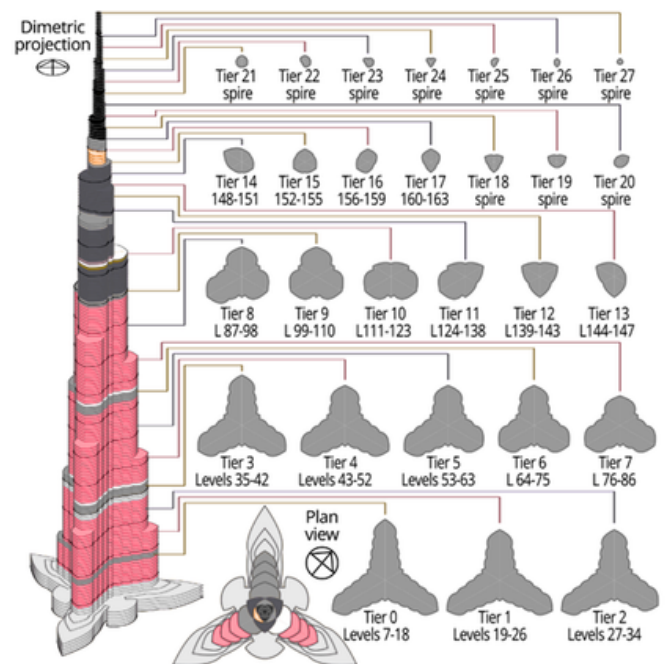


Fig 2 Schematic diagram of the Burj Khalifa's buttressed core structural system, showing the hexagonal core, buttressing wings, and outrigger connections.

design principle, ensuring that if any single structural element is overstressed, alternative load paths can safely redistribute forces, preventing progressive collapse. The system is designed for **controlled flexibility**; the high strength reinforced concrete elements were

detailed per **ACI 318-02** standards for **special moment frames**, ensuring sufficient ductility and shear capacity for energy dissipation during cyclic seismic loading. This allows the structure to **absorb and dissipate significant seismic energy** without catastrophic failure. The tower rests on a **pile-supported raft foundation**: a **3.7-meter-thick reinforced concrete mat**, constructed in four separate pours. This massive raft is supported by **192 bored, cast-in-place reinforced concrete piles**, each **1.5 meters in diameter and forty-three meters long**. These piles penetrate the weak surface sandstone and siltstone layers to anchor the structure in deeper, stable strata.

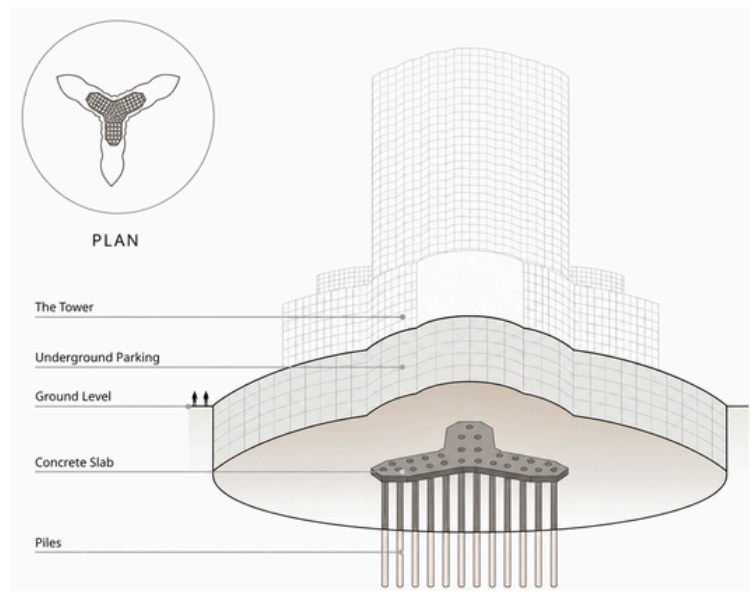


Fig 3 Pile-supported raft foundation

Addressing Seismic Hazards: This **pile-supported raft** foundation was engineered to mitigate two primary geotechnical-seismic risks: **liquefaction potential** in the weak upper sandstone/siltstone layers and **differential settlement**. The 43-meter-long piles transfer the building's enormous load (approximately 500,000 tonnes) to deeper, stable strata, effectively anchoring the structure. This prevents the dangerous tilting or uneven settlement that could be triggered or worsened by seismic shaking, ensuring the superstructure's vertical alignment and integrity are supported during a seismic event.

330,000 cubic meters of high-performance concrete, with specified strengths of **C60 and C80 grade**, providing exceptional compressive strength and durability. Combat Dubai's extreme heat and prevent premature curing and thermal cracking, which could compromise seismic performance concrete was poured at night using chilled water and ice in the mix.

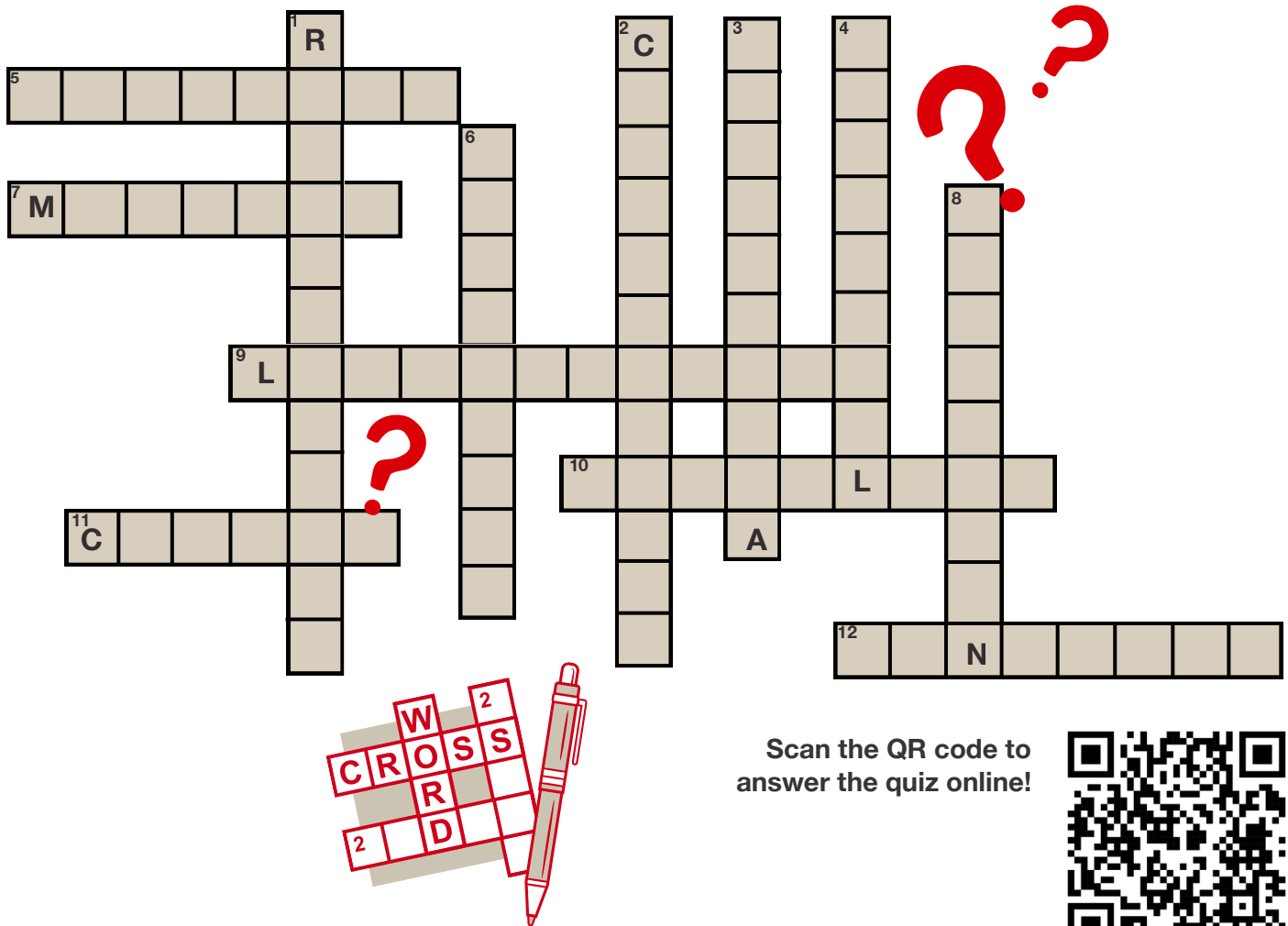
The central core was built using a **jump-form construction** system, a self-climbing formwork technique. This method was vital for ensuring the monolithic integrity, perfect verticality, and uniform quality of the primary lateral-force-resisting system, dropping potential weak planes.

Real-Time Structural Health Monitoring (SHM): A sophisticated SHM system provides continuous data on the building's behaviour. This system tracks long-term effects like creep and settlement and serves as a critical diagnostic tool, capable of detecting any anomalous shifts or vibrations, thereby confirming design performance under all conditions, including seismic events.

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DOWN

- 1: Upgrading an existing structure for improved seismic performance.
- 2: New Zealand city referenced for its devastating earthquake.
- 3: 1989 California earthquake referenced in the journal. (Use without the space.)
- 4: Type of irregularity caused by uneven mass or stiffness distribution.
- 6: Strengthening technique using additional concrete or steel around members.
- 8: Base _____ reduces seismic forces transmitted to a building.

ACROSS

- 5: Modern seismic engineering aims for rapid post-earthquake _____.
- 7: Brick or stone construction
- 9: Soil phenomenon that causes ground to lose strength during earthquakes.
- 10: Ability of a structure to deform without sudden failure.
- 11: Type of loading used to test seismic anchor performance.
- 12: HIT-Shear technology improves _____ shear resistance.



ANSWER-KEY

Rohan Vashisht, IIT-Hyderabad
Geeta Surashe, S R consultant

DOWN:

1. BRACING
2. FREEBOARD
4. MAGNITUDE
6. SKYBRIDGE
7. DISPLACEMENT
11. PILES

ACROSS:

3. DIAGRID
5. SLOSHING
8. ISOLATION
9. DHAJJIDEWARI
10. PIER
12. DUCTILITY

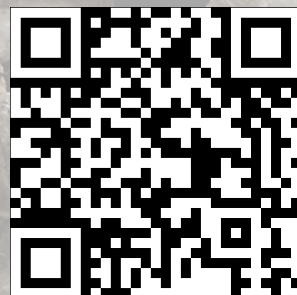


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