

A Comparative Analysis of Lateral Load Resisting Systems for Optimal Dynamic Response in Refinery Steel Structure

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Abstract - Earthquake is such a devastating natural disaster that its likelihood of occurrence or the severity of ground motion it creates still eludes mankind. Refinery steel structures are one of the key civil infrastructures vulnerable to this natural disaster. It poses unique challenge to develop effective Lateral Load Resisting Systems (LLRS) for these structures considering its mass and geometric irregularities, vibrations, spread and mid-rise heights, seismic design code demands and more. Per the provisions of IS 1893:2016, a comparative dynamic analysis is performed on two identical specimens of refinery steel structures conjoined with distinct Special Concentrically Braced Frames (SCBF) LLRS: (a) Two-storey spilt X-brace and (b) Standard X-brace. Applying the philosophy of capacity-based ductile design of steel structures expounded in IS18168:2023, the first phase of the analysis optimizes the member sizes per IS 800:2007 and IS18168:2023 code provisions with considerations for P-delta effects and Modal Response Spectrum Analysis (MRSA) demands in Staad.Pro. The second phase of the analysis evaluates the capacity of these consolidated specimens through Nonlinear Static Pushover Analysis in Staad.Pro with considerations for FEMA 356-2000 acceptance criteria for Life Safety (LS) level. The optimal specimen is perceived through comparing key performance indicators (KPIs) formulated from the analysis results.

Key words: Dynamic Analysis, LLRS, SCBF, MRSA, Pushover

I. INTRODUCTION

The aftermath of a devastating earthquake is so intense that it affects all walks of life in the affected region. It is highly unpredictable to pinpoint the onset of a seismic shaking nor its magnitude. Mankind's risk mitigation strategy with this high-stake natural disaster often revolves around either risk reduction or risk acceptance with reduced damage to life and property. Given the sweeping societal and economic impacts of seismic hazards, nations around the world continue to upgrade their building code provisions. Engineers often follow FEMA 356-2000 regarding safety guidelines in seismic design and retrofitting.

Post Northridge earthquake, seismic design is more a matter of proportioning the structure for strength versus ductility [1]. American Institute for Steel Construction (AISC) has put forward the philosophy of ductile design and the strategy of strong column weak beam capacity-based failure mechanism. As per the AISC Education [2], ductility is the ability to deform in the inelastic range of a material. So far, steel remains the best construction material for this. Conforming to these developments, Indian seismic codes IS1893(part 1): 2016 and IS18168: 2023 have brought several changes to the seismic design provisions. Per IS18168: 2023, admissible seismic lateral load resisting systems are Special Moment Resisting Frames (SMRF), Special Concentrically Braced Frames (SCBF) and Eccentrically Braced Frames (EBF). These systems are associated with response reduction factors, represented by R to induce the intended ductility to the structure.

The structure is expected to yield and develop plastic hinges which reduces the potential damage caused by sudden brittle failure [4]. It is essentially a risk reduction strategy where moderate damage to the structure is accepted at the same time safeguards the inhabitants and valuables. However, the probability of occurrence is made very rare by code prescribed values of Maximum Considered Earthquake (MCE) for the region. As per the code [3], it is associated with a 2,475-year return period and approximately 2% probability of exceedance in 50 years. Different structural types utilize distinct lateral load-resisting configurations to optimize performance. High-rise residential buildings typically use architectural shear walls and central cores, whereas industrial buildings rely on vertical bracing for lateral load resistance. Among these, refinery steel structures are characterized by irregular shape, irregularities in mass, stiffness and geometry, moderate heights, stringent code compliance, and are not often architecturally built. Its form and function are aligned to the specific refinery process flow. Consequently, selecting an effective lateral system that ensures both adequate strength and code compliance is a unique challenge.

A quantitative research methodology is chosen to find out an optimal lateral load resisting system for an idealized refinery steel structure. For this, two identical specimens are modeled but equipped with distinct lateral load resisting systems. Using descriptive research approach, dynamic analysis is performed on the specimens with the aid of

STAAD.Pro software. The analysis covers geometric nonlinearity of the structure using P-Delta analysis followed by Modal Response Spectrum Analysis (MRSA) procedure to consolidate member sizes in the first phase. In the second phase, seismic evaluation is performed using Nonlinear Static Pushover Analysis to obtain the best-fit specimen.

II. LITERATURE REVIEW

The literature review for this thesis covers a wide range of topics, which are briefly reviewed below for the various pros and cons and the gaps that emerge.

A. Effect of Soil and Foundation:

The soil structure interaction is a major development in the revised IS1893(part 1):2025 seismic provisions. The conventional rigid foundation concept has paved way for considering soil flexibility in foundation design, especially for taller buildings over 150m tall. As per the code [5], this can lead to significant reduction in design base shear. Detailed soil investigation is mandatory as per code provisions [5] for seismic zone 5. Cavalieri et al. [6], in their study of dynamic soil structure interaction for fragility characterization of shallow foundations, used unreinforced masonry structures as specimen. Faisal et al. [7], have detailed the effect of soil structure interaction on the dynamic response of RCC building irregularities and found that fundamental period, base shear and storey drift are affected by non-linear dynamic response of the building for varying soil bearing capacities. Refinery structures are characterized by deep foundations such as pile and caissons or even drilled shafts and the effect of modelling soil as an elastic spring for the various foundations above itself is a broad research arena. It is generally mandated in high end nonlinear dynamic analysis procedure based on performance-based design philosophy.

B. Choice of Material:

The ductile design philosophy propagates strong column-weak beam capacity-based failure mechanism strategy. For this approach, steel suits well choice owing to its immense ductility. However, the use of metal matrix composites to induce more ductility and effective corrosion resistance is a new development in the literature. Fahmy & Ibrahim [8] studied the effect of steel-FRP reinforced concrete moment resisting frames subjected to lateral loads. The study [8] found higher deformability and lower damage levels in these composites which are plausible results. Alterations to conventional steel sections, studied by Alish & Wagh [9] found that replacing uniform column sections with laced/battened columns are more cost effective. When it comes to refinery steel structures, not only seismic effects but also, effects of wind loads, impulsive and transient blast resistance loads, machine vibrations, fire resistance etc. have key impact on the overall structural design and steel is an unequivocal choice.

C. Building irregularities:

Irregularities can be in the form of mass, stiffness or geometry and can induce detrimental torsional effects that

must be accounted for in the dynamic analysis of the structure. The placing of boilers piercing through multiple floors, drop zones, multi-floor equipment etc. in refineries can induce significant horizontal and vertical irregularities. Also, structural adjustments from regular shape to accommodate the smooth process flow of refinery operations bring in significant geometric irregularity. Generally, modal mass participations are inevitably reduced due to these irregularities which is a major concern. Akhildev et al. [10], studied the dynamic response of building with soft storey, a type of vertical irregularity, using Response Spectrum Analysis (RSA). Soft stories are highly vulnerable to seismic effects and are one of the major causes of added storey drift [1]. Mali & Patil [11] studied the effect of various types of lateral load resisting systems for different geometric shapes of high-rise buildings, considering both wind and seismic loads. However, the choice of the LLRS is mandatorily confined to SCBF, SMRF or EBF when it comes to seismically vulnerable high-stake structures like refineries [5].

D. Variations in LLRS versus Choices:

The code [5] provisions mandate SCBF, SMRF or EBF for seismic lateral load resisting systems of industrial structures. Innovative use of systems such as Outrigger and Belt Truss, Tubular Central Core etc. are exposed in new research papers. Bagul et al. [12], studied the optimized use of space to reduce obstruction using outrigger and belt truss system. While this is a plausible solution for refinery steel structures, the space constraints and its management need to go along with the smooth process flow of refinery which is of primary concern.

E. Analysis Method and Software Choice:

One of the co-creators of the Response Spectrum Method, Wilson [13] now considers it outdated citing that it was originally used to find random vibrations of mechanical equipment to produce a maximum peak value for one force or one displacement at an unspecified point of time. Whereas, the SRSS and CQC method used in the RSA needs additional approximations when it comes to seismic vibration which are of very short durations [13]. Santosh et al. [14] studied the effective placing of LLRS and its impact on high rise RCC frames using RSA method. The study [14] was performed for LLRS on RCC structures which has significantly different ductility compared to steel structures. Nonlinear Time History Analysis [15] procedure could have been a better choice although it is laborious and requires validation of the procedure by an independent engineering team as per FEMA-356 guidelines. For industrial steel buildings with special concentrically braced frames, STAAD.Pro provides highly versatile nonlinear static Pushover Analysis which delivers more accurate performance outcomes.

F. Structural Optimization:

Significant to seismic designs are the multiple acceptable analysis methods both in the linear and nonlinear streams. The results of each of these methods varies, from

conservative design to overstrength capacity-based design. Do Carmo [16] in the doctoral research, explored a comparison between the deterministic and probabilistic structural optimization using the power spectral density function under time varying loads. Mahdizadeh & Gholizadeh [17] studied the seismic design optimization of steel moment frames by neural networks and found this technique reasonable and cost effective. Particle Swarm approach for structural optimization was studied by Perez & Behdinen [18]. Significance of linear programming optimization by Kulhari [19] found it an indispensable tool for modern day dynamic optimization.

G. Summary of Research Gaps:

To summarize, the seismic characterization of refinery steel structures poses unique challenge in terms of irregularities, ductility of structure, choice of LLRS and its analysis methods, software preference to capture the nonlinear dynamic response, soil structure interactions, and above all the persistently evolving seismic provisions. Attempts to capture all these research gaps are given importance in the current study.

III. METHODOLOGY

A. Research Design:

The aim of this dissertation is to find the optimal LLRS among two different Special Concentrically Braced Frame LLRS: Two-storey split X-brace & Standard X-brace. For this, two identical idealized refinery steel structure is modelled [20] and the above LLRS is fitted on each of them. Dynamic analysis [21, 22] is performed on these specimens under the provisions of IS 1893 (part 1):2016, IS 1893 (part 4):2024 and IS 18168:2023 in conjunction with IS800:2007 steel design code. Also, FEMA 356-2000 [23] guidelines are incorporated through Pushover Analysis for the seismic evaluation of the LLRS specimens.

A quantitative research methodology is followed to achieve this aim, wherein a descriptive research approach to observe, record, and summarize the characteristics of the variables through structural engineering software analysis is adopted. These characteristics are formulated as Key Performance Indicators (KPIs) from the analysis results. Finally, by comparing the KPIs among the specimens, the best-fit specimen is chosen.

The study applies STAAD.Pro's Modal Response Spectrum Analysis (MRSA) [24] and Nonlinear Static Pushover Analysis [25] methods to understand the characteristics of the LLRS. Both are dynamic seismic analysis procedures recommended by the respective Indian seismic codes [3].

The philosophy of capacity-based ductile steel design forms the background knowledge. According to this philosophy, a structure safely deforms and absorbs extreme seismic energy without collapsing, rather than relying solely on high strength or rigidity [5, 25].

While this dissertation focuses on analysis for best-fit LLRS, ductile design of the whole structure is reserved for future research and is beyond the scope of this study.

B. Data Collection: General Building Parameters

- Type of structure: Idealized structural steel refinery structure (Fig 1)
- Structure Size: 12m L x 12m B x 30m H (Midrise Structure)
- Bay length: 2 bays @ 6m each full height in both x & y directions, (Fig 2).
- LLRS choice: SCBF (Covered in section D for specimen details).
- Seismic category of the structure: I (Process Building with Hazardous Chemical Industrial Facility) [3], Importance Factor, $I=1.5$
- Seismic Zone: IV (Fig 9), Severe intensity zone as per IS1893(part 1): 2016.
- Location: Kochi (For wind load calculations)
- Refinery features:
 - Metal deck roofing and side cladding
 - Flooring: 150mm thick composite deck concrete floor
 - Floor access: Metal staircase, ladders & Elevator up to full height
 - Floor perimeter enclosure: Pipe handrails
 - Ventilation: Louvre system (1m deep on all floor perimeters) (Fig 1)

C. Specimen Parameters:

This thesis is performed as part of M-Tech Structural Engineering Dissertation spanning final two semesters. Specimens are chosen based on how efficiently they can be modeled and evaluated within the project's time constraints. IS18168:2023 code provisions and seismic zone selected mediate the choice of LLRS.

D. LLRS Choice:

- SCBF-1: Two-storey split X-brace (Fig 3)
- SCBF-2: Standard X-brace (Fig 6)

E. Analysis Specimens:

Each of the SCBFs listed above is conjoined to the idealized refinery steel structure skeleton to form two separate analysis specimens.

- Specimen-1 – SCBF_2XB - Refinery Steel Structure w/two storey split X-brace (Fig 4)
- Specimen-2 – SCBF_1XB - Refinery Steel Structure with standard X-brace (Fig 7)

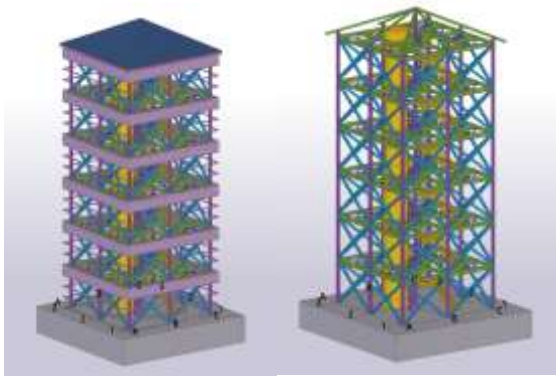


Fig-1: (left) Idealized Refinery Steel –View All
Fig-2: (right) Idealized Refinery Steel –View Main
(Fig 1 & 2 prepared in Tekla Structures)

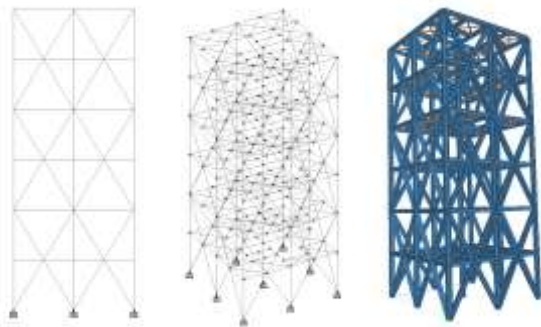


Fig-3: (left) LLRS choice 1 - SCBF – 1: Two-storey split X-brace STAAD Pro elevation

Fig-4: (middle) Specimen-1: SCBF 2XB - STAAD Pro model

Fig-5: (right) Specimen-1 STAAD Pro 3D Rendered View

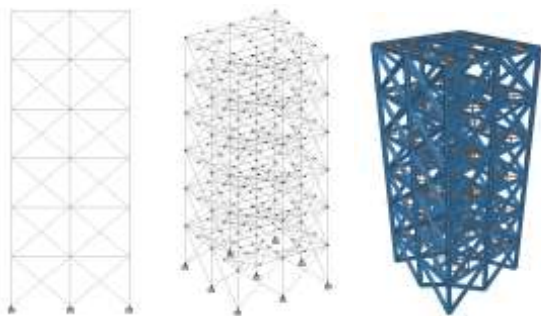


Fig-6: (left) LLRS choice 2 - SCBF – 2: Standard X-brace STAAD Pro elevation

Fig-7: (middle) Specimen-2: SCBF 1XB - STAAD Pro model

Fig-8: (right) Specimen-2 STAAD Pro 3D Rendered View

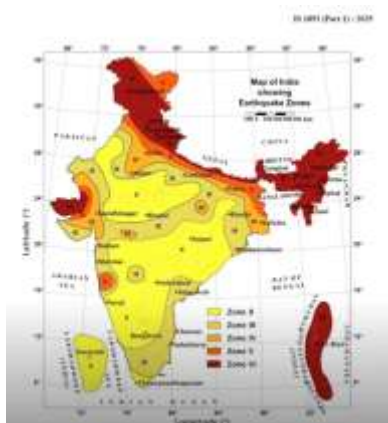


Fig-9: Seismic Zones in India Map (Courtesy IS1893 (Part1): 2025)

F. Ethical Considerations:

Idealized refinery steel skeleton is arrived at based on referring conventional refinery structures in India and

abroad which are active in service and simplified for the feasibility of the study.

G. Limitations of the study:

- The scope of this study is strictly bounded by the academic M. Tech in Structural Engineering dissertation timeline. Hence, seismic zone 4 is selected [5]) instead of the extreme zones 5 or 6, which require more rigorous nonlinear dynamic analysis.
- Reflecting the research focus, seismic forces are prioritized over wind forces as predominant lateral loads. However, static load combinations including wind load, dead load and live load are included for performing geometric nonlinearity P-Delta effects on the structure prior to Modal Response Spectrum Analysis.
- For simplifying calculations, only main steel self-weight and floor loads are considered in the analysis. Vibration effects, blast resistance and other impulsive transient load effects are not considered.

IV. STRUCTURAL ANALYSIS

A. Preliminary Inputs:

The preliminary inputs for the analysis cover both refinery specific and seismic specific characterizations forming part of the idealized structure.

(1) Refinery steel characterization

- Beams, columns and LLRS braces – WPB profiles as per IS18168:2023
- Horizontal bracing: Double angles w/ long leg vertical & back-to-back
- Material grades: E250 (For Ductile Design approach)
- $F_y = 250 \text{ kN/m}^2$ $F_u = 410 \text{ kN/m}^2$
- Modulus of Elasticity of Steel, E : 210 GPa
- Density of steel, ρ : 7850 kg/m^3 (76.819 kN/m^3)

(2) Seismic characterization of the structure

- Overall height of the structure (H): 30m (mid-rise structure)
- Plinth ($L \times B$): $12\text{m} \times 12\text{m}$ - ($6\text{m} \times 2$ bays each side)
- Building aspect ratio (H/B): $30/12 = 2.5$
- The aspect ratio of 2.5 makes the structure moderately slender.
- Overturning Moments: High axial force variations occur in boundary columns due to lateral earthquake actions.
- P-delta effects: P-delta geometric non-linearity check required if the lateral drift is significant (exceeds 0.4%) as per IS1893(1):2016. Refer chapter V for details.

➤ Building Irregularities: Refer Fig 10 & 11 for Building Plan and Elevation views.

- Horizontal Irregularity: Present, since boiler unit, drop zone, ladder access, elevator access and stair access require floor openings.
- Vertical Irregularity: Nill, uniform plinth considered.
- Torsional irregularity: Check required after analysis, covered in section V.
- Re-Entrant Corners: Nill (Fig 2)
- In plane/Out of plane Discontinuity& Floating Columns: Nill
- Mass Irregularity: Uniform floor mass considered.
- Weak storey/soft storey effect: NA, uniform stiffness and storey heights considered.

(3) Criteria for Selection of Dynamic Analysis: (IS1893(1):2016)

- Dynamic analysis required since building irregularities are present.
- Dynamic analysis required since the building is moderately slender with aspect ratio = 2.5
- Research Specific considerations: Modal Response Spectrum Analysis preferred for dynamic analysis.
- Analysis Software preference: STAAD.Pro Connect Edition
 - Modal Response Spectrum Analysis with P-delta considerations for seismic analysis of LLRS
 - Non-Linear Static Pushover Analysis for seismic evaluation of LLRS

C. Modelling for Seismic Analysis

Building functional plan & elevation views are shown below in Fig 10 & 11. The beam-column connections are considered fully rigid for Pushover Analysis [25], the SCBF braces form the fuse of the structure [26] as per the capacity-based ductile design philosophy [4].

Other than SCBF braces, structural elements such as collectors, columns and horizontal rigidity bracing and its connections are designed for overstrength (Ω_0) load combinations given in IS18168:2023 while the SCBF braces are having standard load combinations per IS800:2007 & IS1893(part 1):2016. Fixed base followed for all columns. Connection design and foundation design are not included in scope.

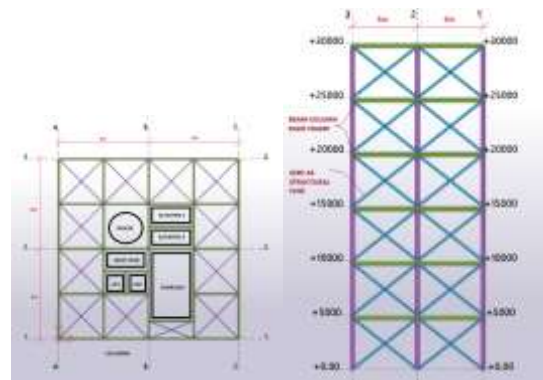


Fig-10: (left) Building floor plan (Typical at all floors)
Fig-11: (right) Building Elevation view (shown with Specimen-2)
(Fig 10 & 11 prepared in Tekla Structures)

D. Load Path settings

- Floor Diaphragm: 150mm composite deck concrete floor slab
- Perimeter horizontal bracing at all floors - for torsional rigidity, transferring loads to collector beams [27]
- Diaphragms to collector beams
- Collectors to LLRS
- LLRS to ground through columns

E. Load calculations:

(1) Building codes:

- IS800:2007, IS875 (Part 1, 2 & 3)
- IS1893(1):2016, IS1893(4):2015
- IS18168:2023

(2) Dead Load (DL):

- Self-weight of the members
- Floor load (for 150mm thick concrete slab) = 3.75 kN/m^2
- Floor finishes = 1.25 kN/m^2

(3) Imposed Load (IL): Denoted as live load (LL) here onwards.

- Floor load = 5 kN/m^2 for industrial structures

(4) Wind Load calculations (WL): As per IS 875 Part 3:2015

- Basic Wind Speed (V_b)
- Structure location: Kochi (Kerala) lies (Zone III) with $V_b=39 \text{ m/s}$ (from IS 875 wind map). However, for seismic calculations, Zone IV considered.
- Design Wind Speed at Height z (V_z)
 - $V_z = V_b \times k_1 \times k_2 \times k_3 \times k_4$ ----- (1)
 - k_1 Risk Coefficient = 1.0 (50-year design life for industrial building).
 - k_2 Terrain, Height & Size Factor:
 - Kochi urban/industrial → Terrain Category 3.
 - At 30 m height, $k_2 \approx 1.06$.
 - k_3 Topography Factor = 1.0 (flat terrain).

- k4 Cyclonic Importance Factor = 1.15 (Kochi is in cyclonic-prone coastal region).
- $V_z = 39 \times 1.0 \times 1.06 \times 1.0 \times 1.15 \approx 47.5 \text{ m/s}$
- Design Wind Pressure (P_z)
 - $P_z = 0.6 \times V_z^2 \text{-----(2)}$
 - $P_z = 0.6 \times 47.541^2 \approx 1.356 \text{ kN/m}^2$
- Net Wind Pressure on Surfaces
 - $F = P_z \times A \times (C_{pe} - C_{pi}) \times K_d \text{-----(3)}$
 - A = Projected area (e.g., wall = $30 \text{ m} \times 12 \text{ m} = 360 \text{ m}^2$).
 - C_{pe} = External pressure coefficient (walls: ± 0.8 , roof: ± 0.7 to ± 1.2 based on slope).
 - C_{pi} = Internal pressure coefficient (± 0.2 for large openings).
 - K_d = Directionality factor (0.9 for frames).

$F = 1.36 \times 360 \times (0.8 - (-0.2)) \times 0.9 \approx 440.64 \text{ kN}$ (windward wall)

(5) Seismic Load calculation (EQ):

As per IS1893(1):2016, equivalent SDOF system with lumped mass at floors (6 floors) considered.

- Key Parameters from IS 1893:2016
 - Seismic Zone (Z): 0.24 (Zone IV)
 - Importance Factor (I): 1.5 (Table 6 of IS 1893) - High importance
 - Response Reduction Factor (R): For SCBF, R = 4.5
 - Soil Type: Hard soil → Average Response Acceleration Coefficient (S_a/g) taken from spectrum curves given in the code.
 - 5% Accidental eccentricity from centre of mass on both sides of the floor plan dimension perpendicular to the direction of the earthquake.

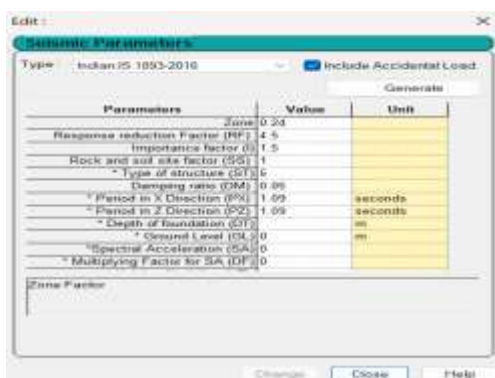


Fig-12: Seismic Definitions in STAAD.Pro for the specimens

- Building Geometry & Seismic Mass
 - Plinth: $12 \text{ m} \times 12 \text{ m}$, height = 30 m
 - Floors 1–6: $12 \text{ m} \times 12 \text{ m}$, height = 5 m each
 - Floor System: 150 mm composite concrete slab
 - From Dead Load (DL)
 - Slab self-weight = $0.15 \text{ m} \times 25 \text{ kN/m}^3 \approx 3.75 \text{ kN/m}^2$
 - Floor finishes = 1.25 kN/m^2
 - Total slab weight = $3.75 + 1.25 = 5.0 \text{ kN/m}^2$
 - Steel framing self-weight (approx.) $\approx 2.0 \text{ kN/m}^2$ (Varies with final design)
 - Total DL $\approx 7.00 \text{ kN/m}^2$

- From Live Load (LL)
 - Industrial use → 5.0 kN/m^2 (per IS 875 Part II)
 - As per IS 1893, 50% of LL considered for seismic weight.
 - Seismic Weight per Floor
 - All floors ($12 \times 12 = 144 \text{ m}^2$): $W = (7 + 2.5) \times 144 \approx 1,368 \text{ kN}$ per floor
- Fundamental Natural Period (T)
 - For steel braced frames:
 - $T = 0.085 \times h^{0.75} \text{-----(4)}$
 - where $h = 30 \text{ m}$
 - $T = 0.085 \times 30^{0.75} \approx 1.09 \text{ s}$
- Design Horizontal Seismic Coefficient (A_h)
 - $A_h = Z/2 \times I/R \times S_a/g \text{-----(5)}$
 - $Z = 0.24$, $I = 1.5$, $R = 4.5$, $S_a/g \approx 2.5/T = 2.5/1.09 \approx 2.29$ (capped at 1.0 for hard soil spectrum).
 - Using $S_a/g = 1.0$.
 - $A_h = 0.242 \times 1.56 \times 1.0 = 0.04$
- Design Base Shear (V_b)
 - $V_b = A_h \times W \text{-----(6)}$

Total seismic weight $W = 6 \times 1368 = 8208 \text{ kN}$.

$V_b = 0.04 \times 8208 = 328 \text{ kN}$.

Vertical Distribution of Seismic Forces (IS 1893:2016)

$$Q_i = \frac{V_b \cdot W_i h_i}{\sum W_j h_j} \text{-----(7)}$$

Computing $W_i h_i$ for each floor (weights $\times$ heights) & distributing 328 kN proportionally. Here, Plinth, storey height and floor slab depth are same for all floors. Therefore, each floor receives almost same mass, mass irregularity neglected.

F. Load combinations: Values are read from previous section

- For P-Delta Analysis (IS800:2007 – Static Repeat load in STAAD.Pro)
 - $1.2 \text{ DL} + 1.2 \text{ IL} + 1.2 \text{ WL}$
 - $1.5 \text{ DL} + 1.5 \text{ WL}$
 - $0.9 \text{ DL} + 1.5 \text{ WL}$
 - $\text{DL} + 0.8 \text{ IL} + 0.8 \text{ WL}$
 - $\text{DL} + 1.0 \text{ WL}$
- For Modal Response Spectrum Analysis (IS800 + IS1893)

Combination Group A: $1.2(\text{DL} + \text{IL} \pm \text{EL})$

- $1.2\text{DL} + 1.2\text{IL} \pm 1.2\text{EL}_X \pm 0.36\text{EL}_Z$
- $1.2\text{DL} + 1.2\text{IL} \pm 1.2\text{EL}_Y \pm 0.36\text{EL}_Z$
- Combination Group B: $1.5(\text{DL} \pm \text{EL})$
 - $1.5\text{DL} \pm 1.5\text{EL}_X \pm 0.45\text{EL}_Z$
 - $1.5\text{DL} \pm 1.5\text{EL}_Y \pm 0.45\text{EL}_Z$
- Combination Group C: $0.9 \text{ DL} \pm 1.5 \text{ EL}$
 - $0.9\text{DL} \pm 1.5\text{EL}_X \pm 0.45\text{EL}_Z$
 - $0.9\text{DL} \pm 1.5\text{EL}_Y \pm 0.45\text{EL}_Z$
- Combination Group D: $1.0 \text{ DL} + 0.8 \text{ IL} \pm 0.8 \text{ EL}$
 - $1.0\text{DL} + 0.8\text{IL} \pm 0.8\text{EL}_X \pm 0.24\text{EL}_Z$
 - $1.0\text{DL} + 0.8\text{IL} \pm 0.8\text{EL}_Y \pm 0.24\text{EL}_Z$
- Combination Group E: $1.0 \text{ DL} \pm 1.0 \text{ EL}$
 - $1.0\text{DL} \pm 1.0\text{EL}_X \pm 0.30\text{EL}_Z$
 - $1.0\text{DL} \pm 1.0\text{EL}_Y \pm 0.30\text{EL}_Z$

- Overstrength Load Combinations (IS800 + IS1893 + IS18168)

These are in addition to the above load combinations, for overstrength design of force-controlled elements once LLRS member sizes are consolidated under standard load combinations. All members and its connections other than SCBF are part of force-controlled elements in the specimens and receives this overstrength load combination.

$$1.2DL + \gamma_{LL}IL \pm 1.0EL_m$$

$$0.9DL \pm 1.0EL_m$$

γ_{LL} = partial safety factor for live load (typically 0.25 or 0.50 depending on storage/occupancy load criteria as aligned with seismic mass)

$$EL_m = \Omega_o \times EL = 2.5EL$$

where Ω_o = overstrength factor for SCBF = 2.5

- For Nonlinear Static Pushover Analysis (IS800:2007 – Gravity Load)

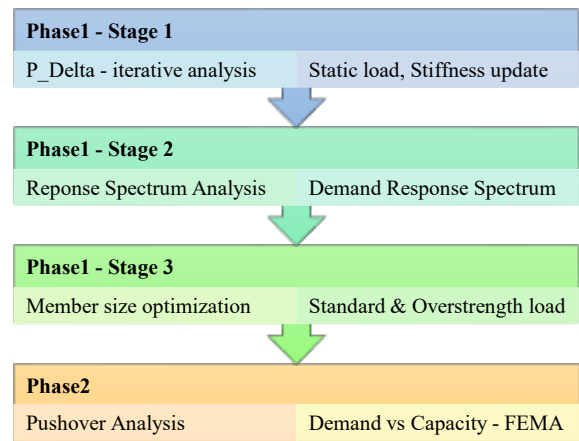
DL+IL combined Gravity Load considered. Two methods of vertical distribution of pushover load followed: uniform pattern and modal pattern as per FEMA-356, which are covered in Section 4.7.3.

G. Analysis Work Flow

The analysis is split into two phases: phase 1 for seismic analysis and phase 2 for seismic evaluation.

- In phase 1, seismic analysis is performed. Initially P-Delta second order effects on static loads (DL, LL & WL) is performed to check the geometric nonlinearity of the structure since aspect ratio is 2.5. This updates the stiffness of the structure followed by Modal Response Spectrum Analysis and member design optimization. The following two groups of load combinations are used:
 - Standard load combination for the deformation controlled LLRS brace elements as per IS800 & IS1893, which acts as the fuse of the structure.
 - Overstrength load combinations for the force-controlled elements (all structural elements other than SCBF). As per IS18168:2023, columns to remain elastic at the same time strong column - weak beam principle applies [4].
- In phase 2, seismic evaluation using pushover analysis is performed on the consolidated specimens from phase 1 to understand the seismic demand versus capacity. See Chart 1 for quick reference.

Chart 1: Analysis Flow Chart



H. Seismic Analysis parameters in software

The following seismic analysis parameters are followed with respect to STAAD.Pro definitions for the same and in conjunction with the analysis work flow in the previous section.

(1) Parameters for P-delta Analysis

P-Delta analysis is an iterative second-order analysis used to account for the additional bending moments caused by gravity loads acting on the laterally displaced structure. The following parameters and options are considered:

- Iterations (n): 20 considered. Specifies the maximum number of iterations STAAD.Pro will perform to reach convergence.
- Delta Effects: Small Delta / Both: Both considered. Calculates both Large Delta (global sway of the structure) and Small Delta (local bending or flexural buckling of individual members between braced points).
- Geometric Stiffness (K_G Matrix): Not considered

(2) Parameters for Modal Response Spectrum Analysis

To perform this analysis, both global dynamic parameters and specific excitation/load parameters need to be defined. The following parameters from the software are used:

- Direction (X, Y, or Z): X considered; columns aligned as H in global X direction.
- The 30% orthogonal rule in load combination (100% in one direction + 30% in the perpendicular direction) is already automatically added inside the solver when both X and Z are checked in the same dialog box.
- Scale Factor: The multiplier used to scale the spectral values to the units and design intent. It is automatically set to 1.0 when IS1893:2016 selected, for example base shear value from the solver is matched to empirical base shear value calculated using Equivalent Static Load method.

- Damping Ratio (DAMP): Defines the viscous damping of the structure (e.g., 0.05 for 5% damping) used for spectrum generation.
- Mode Combination Method: CQC (Complete Quadratic Combination)
- Seismic code: IS1893(1):2016)
- Seismic Zone: Zone 4
- Importance Factor (I): 1.5, Factor depending on the functional use of the building.
- Response Reduction Factor (R): 4.5 for SCBF, Factor for the ductility and energy-absorbing capacity of the lateral force-resisting system.
- Soil Type / Site Class: Hard Soil considered
- Cut-off Frequency / Number of Modes: 100 chosen to adequately capture 90% mass participation.
- Mass Source: Gravity load case created, tells the program which weights/masses to convert into dynamic masses (often includes 100% Dead Load + a percentage of Live Load.

Target design response spectrum: Based on the above parameters, target design response spectrum is prepared in Fig 13.

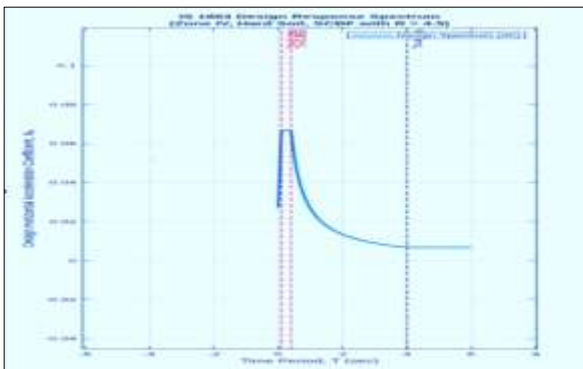


Fig-13: Target Design Response Spectrum (MATLAB graphs)

(3) Parameters for Nonlinear Static Pushover Analysis

Static pushover analysis evaluates seismic performance through nonlinear load-deformation behaviour using the Displacement Coefficient Method (per FEMA 356). The basic parameters given below define how the structure is loaded, how it yields (hinges), and when the analysis stops.

- Loading Pattern: FEMA 356 recommends checking more than one load pattern. Software allows any one method at a time. Hence, Method 3 is preferred. However, a comparison of the relative merits of both methods for the specimens are included in Table 5
- Method 1 – Parabolic Distribution
 - Calculates the vertical distribution vector proportional to the product of floor weight and height raised to an exponent k .

- The lateral force at floor x is: $F_x \propto w_x h_x^k$
- w_x = seismic weight at level x
- h_x = height of level x from base
- k = based on fundamental period
- Since 30m tall structure has a fundamental period ($T > 0.7s$), the exponent is set to ($k = 2.0$). This transforms the loading vector into a steep parabola

➤ Method 3 – Uniform Mass Distribution.

- Method 3 applies a completely uniform, rectangular block of lateral force across all 6 storeys
- The lateral force at each floor is: $F_x \propto w_x$
- Direction: X considered, specifies the direction of the seismic excitation matching mode 1 translation direction.

➤ Pushover Hinges: FEMA 356 defaults covered in section 4.9.

➤ Hinges model the nonlinear load-deformation behaviour of beams and columns, calculates plastic rotation capacities and backbone curve properties according to FEMA standards.

➤ Performance Levels: Sets criteria for damage evaluation: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). The seismic evaluation is performed for Life Safety, covered in section E of chapter V.

➤ Load Steps and Incremental Control:

➤ LD STEP (Load Steps): 250 considered (as required). Determines the number of increments used to apply the push load.

➤ Analysis Limits (Termination Criteria): Displacement at a user defined control joint tells the program when to stop pushing the structure. Initial target displacement set to 0.45m at top floor control joint with coordinates (6, 6, 30).

➤ Performance Point: In chapter V, calculated final target displacement also called as Performance Point is updated from post processing results.

➤ Geometric Nonlinearity: P-delta strength degradation check covered in Table 6.

(4) Backbone curve specifications for Pushover Analysis

The backbone curve (load-deformation curve) for pushover analysis in STAAD.Pro defines the progressive non-linear behavior of plastic hinges. Per FEMA 356 guidelines, this relationship is modeled using a simplified, multi-linear force-deformation spectrum that explicitly charts structural capacity from the initial elastic range down to total failure. Fig 14 shows FEMA-356 Table 5-1

backbone curve for the same. The critical points of this curve are described below.

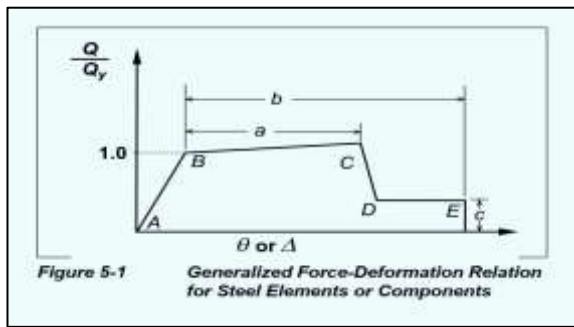


Fig-14: FEMA 356 Table 5-1 - Backbone Curve (Extract)

- Point A (0,0) - Origin: The structural element is completely unloaded and stress-free.
- Point B (1,1) - Nominal Yielding: Represents the effective elastic limit where plastic deformation begins.
- Point C - Ultimate Capacity: The peak strength threshold. The corresponding deformation coordinate is the plastic capacity limit (a), and the force coordinate accounts for expected material strain hardening.
- Point D - Residual Capacity: The drop-off threshold. The corresponding deformation coordinate represents the ultimate plastic limit (b), while the force coordinate defines the residual strength fraction (c) remaining in the member.
- Point E - Total Collapse: The ultimate deformation ceiling. Once a hinge passes Point E, the software strips all structural capacity from the element, forcing a comprehensive cross-frame load redistribution.

Based on the above curve critical points, acceptance criteria are defined as noted below, which is one of governing performance criteria for LLRS brace.

- Elastic Range: From A to B.
- Immediate Occupancy (IO): Between B and C, near the onset of yielding.
- Life Safety (LS): A large post-yield range located between points B and C, showing significant inelastic deformation but safely away from point C.
- Collapse Prevention (CP): Extends just before point C

The acceptance criteria LS is considered for the seismic evaluation of the specimens.

(5) Hinge Specifications for Pushover Analysis

For an SCBF type steel frame, hinges are assigned to both the braces and the beam-column frame elements:

- Brace Members (Axial Hinges): Modelled as truss elements with axial deformation-controlled behaviour. This accounts for tension yielding and

compression buckling properties per FEMA 356 Table 5-7. Compression braces drop drastically in capacity after buckling (Point C to D on the FEMA curve).

- Beams and Columns (Flexural Hinges): Assigned as default FEMA 356 Interacting P-M-M hinges or M-Z moment hinges to columns and beams to capture residual frame action after the braces buckle or yield.

I. STAAD.Pro Editor Command Input

STAAD.Pro command input for the analysis work flows described in section G are given below. When the maximum of two or multiple load groups are required, ENVELOPE option is used to obtain the max value among all groups in the software.

(1) For Phase-1: Seismic Analysis

```
STAAD SPACE
START JOB INFORMATION
ENGINEER DATE xx-xx-xx
END JOB INFORMATION
INPUT WIDTH 79
SET NL 500
UNIT METER KN
JOINT COORDINATES
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
MEMBER INCIDENCES
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
```

```
START GROUP DEFINITION
MEMBER
_BEAMS_COLUMNS
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
_LLRS
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
END GROUP DEFINITION
```

```
START USER TABLE
TABLE 1
UNIT METER KN
WIDE FLANGE
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
```

```
TABLE 2
UNIT METER KN
TUBE
SHS200X200
0.009375 0.2 0.2 0.0125 5.51758e-05 5.51758e-05
8.23974e-05 0.005 0.005
TABLE 3
UNIT METER KN
TUBE
SHS250X250
0.011875 0.25 0.25 0.0125 0.000111947 0.000111947
0.000167456 0.00625 0.00625
```

END
MEMBER PROPERTY INDIAN
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
DEFINE MATERIAL START
ISOTROPIC STEEL
E 1.99947e+08
POISSON 0.3
DENSITY 76.8191
ALPHA 6.5e-06
DAMP 0.03
TYPE STEEL
STRENGTH RY 1.5 RT 1.2
END DEFINE MATERIAL
CONSTANTS
MATERIAL STEEL ALL
SUPPORTS
1 TO 3 22 TO 24 43 TO 45 FIXED
MEMBER TRUSS
* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2
DEFINE IS1893 2016 ACCIDENTAL LOAD
ZONE 0.24 RF 4.5 I 1.5 SS 1 ST 5 DM 0.05 PX 1.09 PZ 1.09 GL 0
CUT OFF MODE SHAPE 100
DEFINE WIND LOAD
TYPE 1 WIND 1
INT 1 1 1.136 1.231 1.293 1.356 1.395 1.434 HEIG 5
10 15 20 25 30 35 40
EXP 1 JOINT 1 TO 135 137 139 141 143 145 147 TO 159 172 TO 183 190 TO 195
DEFINE REFERENCE LOADS
LOAD R1 LOADTYPE Mass TITLE MASS
REFERENCE LOAD
SELFWEIGHT Y -1
FLOOR LOAD
YRANGE 0 30 FLOAD -7.5 XRANGE 0 12 ZRANGE 0 12 GY
END DEFINE REFERENCE LOADS
LOAD 1 LOADTYPE Seismic TITLE EQX
1893 LOAD X 1
LOAD 2 LOADTYPE Seismic TITLE EQZ
1893 LOAD Z 1
LOAD 3 LOADTYPE Dead TITLE DL
FLOOR LOAD
YRANGE 0 30 FLOAD -5 GY
SELFWEIGHT Y -1
LOAD 4 LOADTYPE Live TITLE LL
FLOOR LOAD
YRANGE 0 30 FLOAD -5 GY
LOAD 5 LOADTYPE Wind TITLE WLX
WIND LOAD X 1 TYPE 1 XR 0 12 YR 0 30 ZR 0 12
WIND LOAD -X -1 TYPE 1 XR 0 12 YR 0 30 ZR 0 12
LOAD 6 LOADTYPE Wind TITLE WLZ
WIND LOAD Z 1 TYPE 1 XR 0 12 YR 0 30 ZR 0 2
WIND LOAD Z -1 TYPE 1 XR 0 12 YR 0 30 ZR 0 2
* Repeat Load Generation as per IS:456/IS:800
IS800:2007 TABLE4
PDELTA 20 ANALYSIS SMALLDELTA

CHANGE
MODAL CALCULATION REQUESTED MISSING
MASS
LOAD 34 LOADTYPE Seismic TITLE RSXZ
SELFWEIGHT X 1
SELFWEIGHT Y 1
SELFWEIGHT Z 1
FLOOR LOAD
YRANGE 0 30 FLOAD 5 XRANGE 0 12 ZRANGE 0 12 GX
YRANGE 0 30 FLOAD 5 XRANGE 0 12 ZRANGE 0 12 GY
YRANGE 0 30 FLOAD 5 XRANGE 0 12 ZRANGE 0 12 GZ
SPECTRUM CQC IS1893 2016 X 1 Z 1 DAMP 0.05
SOIL TYPE 1
* Section A: Define Standard Load Combinations (IS 800 + IS 1893)
* These are for standard Beams and Columns
LOAD COMB 101 1.5(DL+LL)
3 1.5 4 1.5
LOAD COMB 102 1.2(DL+LL)+1.2RSXZ
3 1.2 4 1.2 1 1.2
LOAD COMB 103 1.2(DL+LL)-1.2RSXZ
3 1.2 4 1.2 1 -1.2
LOAD COMB 104 1.5DL+1.5RSXZ
3 1.5 1 1.5
LOAD COMB 105 0.9DL+1.5RSXZ
3 0.9 1 1.5
* Section B: Define Overstrength Load Combinations (IS 18168)
* Using $\Omega_0 = 2.5$ multiplier directly on Response Spectrum Forces
LOAD COMB 201 1.2(DL+LL)+2.5RSXZ
3 1.2 4 1.2 1 2.5
LOAD COMB 202 1.2(DL+LL)-2.5RSXZ
3 1.2 4 1.2 1 -2.5
LOAD COMB 203 1.5DL+2.5RSXZ
3 1.5 1 2.5
LOAD COMB 204 0.9DL+2.5RSXZ
3 0.9 1 2.5
* Section C: Concurrent Steel Design Execution
PERFORM ANALYSIS
* Design Block 1: Standard Beams & Columns
*START DESIGN
PARAMETER 1
CODE IS800 LSD
FYLD 250000 ALL
FU 410000 ALL
TRACK 2 ALL
* --- Lateral Stability Parameters ---
* --- Serviceability Parameters ---
LOAD LIST 101 TO 105
CHECK CODE MEMB BEAMS COLUMNS
* Design Block 2: Overstrength Braces (Is 18168)
PARAMETER 2
CODE IS800 LSD
FYLD 250000 ALL
FU 410000 ALL

TRACK 2 ALL

*Strict Seismic/Brace Parameters - Manual check required for IS18168 provisions
LOAD LIST 201 TO 204
CHECK CODE MEMB _LLRS
*END DESIGN

(2) For Phase-2: Seismic Evaluation

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE xx-xx-xx

END JOB INFORMATION

INPUT WIDTH 79

SET NL 500

UNIT METER KN

JOINT COORDINATES

* as per fig 3.4 for specimen-1 and fig 3.7 for specimen-2
MEMBER INCIDENCES

* as per fig 3.4 for specimen-1 and fig 3.7 for specimen-2
DEFINE MATERIAL START

ISOTROPIC STEEL

E 1.99947e+08

POISSON 0.3

DENSITY 76.8191

ALPHA 6.5e-06

DAMP 0.03

TYPE STEEL

STRENGTH RY 1.5 RT 1.2

END DEFINE MATERIAL

CONSTANTS

MATERIAL STEEL ALL

SUPPORTS

1 18 35 48 123 136 169 180 225 238 259 270 293 302

309 318 FIXED

MEMBER TRUSS

* As per Fig 3, 4 & 5 for Specimen-1 and Fig 6, 7 & 8 for Specimen-2

DEFINE PUSHOVER DATA

FRAME 1

GNONL 0

LDSTEP 200

HINGE FEMA ALL

Sl No	Component	Section Type	Limiting Plate Slenderness		
			Web Depth-to-Thickness Ratio	Specimen 1	Specimen 2
1	Beam (Collector)	Doubly-symmetric rolled I-sections	$44.5\epsilon / \sqrt{R_y}$	Pass – See Table 3	Pass – See Table 4
2	Column	Doubly-symmetric rolled I-sections	$(72.7\epsilon / \sqrt{R_y}) * (1 - 1.04Ca)$ for $Ca \leq 0.118$ & $(24.9\epsilon / \sqrt{R_y}) * (2.68 - Ca)$ $\geq (44.4\epsilon / \sqrt{R_y})$ for $Ca > 0.118$	Pass – See Table 3	Pass – See Table 4
3	Brace	Rolled or built-up I-sections	$44.4\epsilon / \sqrt{R_y}$	Pass – See Table 3	Pass – See Table 4

SPECTRUM PARAMETERS

DAMPING 5.0000

SC 4

SS 0.3

S1 0.12

DISP X 0.45 JOINT 63

END PUSHOVER DATA

LOAD 1 LOADTYPE Gravity TITLE GRAVITY

SELFWEIGHT Y -1

FLOOR LOAD

YRANGE 0 30 FLOAD -5 XRANGE 0 12 ZRANGE 0

12 GY

PERFORM PUSHOVER ANALYSIS

FINISH

V. RESULTS

A. Critical Post Analysis Checks

The critical post analysis checks are listed in Table 1. This is mainly from phase 1 of the analysis. Here onwards, specimen with Std X-brace is denoted as 1XB and with Two-storey split X-brace is denoted as 2XB.

Table-1: Checks on Analysis Outputs

Sl No:	Description	Outcome		Remarks
		Specimen1	Specimen2	
		(2XB)	(1XB)	
1	Base Shear multiplier validation	Ok	Ok	Scale 1.0 set
2	Accidental Torsion check	Ok	Ok	Ref: Fig 12
3	Modal Mass Participation - (90% rule)	98.761 (X) 98.817 (Z)	96.790 (X) 96.949 (Z)	Ref: Fig 19
4	Modal Mass Participation - (65% rule)	75.891 (X) 76.142 (Z)	71.470 (X) 71.651 (Z)	Ref: Fig 18
5	Max overall drift = $(0.004 * H)$	0.076 (X)* 0.078 (Z)*	0.080 (X)* 0.081 (Z)*	Max limit = 0.120 m
6	Torsional irregularity - $(\Delta_{max} \geq 1.5 \Delta_{min})$	$\Delta_{max} = 0.076$ $\Delta_{min} = 0.074$	$\Delta_{max} = 0.080$ $\Delta_{min} = 0.079$	Ref: Fig 15
7	Beam weight penalty	Required**	Not required	1XB - advantage

➤ “*” indicates displacement measured at the node where pushover displacement is also measured. It is taken at node with coordinates (6, 6, 30) for the specimens.

➤ “**” indicates force imbalance when one brace buckles for beams located at the midpoint of 2XB braces. Here, 1XB brace is having advantage [28].

B Post Analysis Checks: IS18168:2023 Table 2

The slenderness limit given in IS18168:2023 table 2 are tabulated for the analysis specimens 1 & 2 in Table 2a and 2b for flange and web sections respectively and member sizing tabulated in Tables 3 and 4 respectively. For E250 grade steel chosen, ‘ ϵ ’ factor in IS18168:2023 table 2 uses full capacity and therefore overall weight will be more. In the case of two-storey split X configuration, the brace intersecting beams must also resist brace unbalanced post buckling force [28]. This makes the section heavier on that particular floor leading to potential mass irregularity.

Table-2a: Limiting Width-to-Thickness Ratios for Compression Elements (IS18168:2023 Table 2) (Flange)

Table-2b: Limiting Width-to-Thickness Ratios for Compression Elements (IS18168:2023 Table 2) (Web)

Sl No	Component	Section Type (Choice)	Limiting Plate Slenderness		
			Outstanding Flange Width-to-Thickness Ratio	Specimen 1	Specimen 2
1	Beam (Collector)	Doubly-symmetric rolled I-sections	$9.0\epsilon/\sqrt{R_y}$	Pass – See Table 3	Pass – See Table 4
2	Column	Doubly-symmetric rolled I-sections	$9.0\epsilon/\sqrt{R_y}$	Pass – See Table 3	Pass – See Table 4
3	Brace	Rolled or built-up I-sections	$11.3\epsilon/\sqrt{R_y}$	Pass – See Table5.3	Pass – See Table5.4

where $\epsilon = \left(\frac{250}{f_y}\right)^{1/2}$ and $C_s = \frac{P_u}{P_u/\gamma_{mo}}$, P_u = ultimate design axial load (from analysis results)

Table-3: Summary of Member Sizing - Specimen 1 (2XB)

Structural Component	WPB Section Profile used	IS18168: 2023 mandate (See Table 5.2a & b)
Columns (Floors 1-3)	WPB 450 × 450 (custom built heavy wide flange required)	Axial capacity protection, $KL/r \leq 75$
Columns (Floors 4-6)	WPB 400 × 400 × 191.1 kg/m	Axial capacity protection, stability at upper levels, $KL/r \leq 75$
Braces (2X-type)	WPB 300 × 300 × 88.34 kg/m	To be highly compact profile, to act as fuse, ductile for both tension and compression, $KL/r \leq 160$
Brace intersecting collectors	WPB 600 × 300 × 177.7 kg/m	To resist post buckling unbalanced force, heavier size can create mass irregularity, compensated using uniform sections all floors.
Standard Collectors	WPB 450 × 450 × 91.7 kg/m ((custom built))	Overstrength load transfer without buckling

Table-4: Summary of Member Sizing – Specimen 2 (1XB)

Structural Component	WPB Section used	IS18168: 2023 mandate (See Table 5.2a & b)
Columns (Floors 1-3)	WPB 450 × 450 (custom built heavy wide flange reqd)	Axial capacity protection, $KL/r \leq 75$
Columns (Floors 4-6)	WPB 400 × 400 × 191.1 kg/m	Axial capacity protection, stability at upper levels, $KL/r \leq 75$
Braces	WPB 340 × 300 × 105.0 kg/m	To be highly compact profile, to act as fuse, ductile for both tension and compression, $KL/r \leq 160$
Collectors	WPB 400 × 300 × 124.8 kg/m	Overstrength load transfer without buckling

C. Post Analysis Observations from Phase 1

For specimen 1 & 2 the stipulated torsional irregularity check yields negligible value where $\Delta_{max} \geq 1.5 \Delta_{min}$ for building line dimensions per Clause 7.1, IS1893(1):2016 as seen in Fig 16 & 17

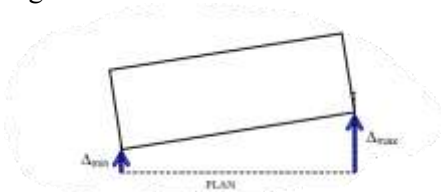


Fig-15: Torsional Irregularity (Ref: IS1893(1):2016)

The modal mass participation percentage remains well over stipulated 90% value for both specimens. The following rules in IS1893(1):2016 govern:

- The 90% Rule: The total sum of modal masses must be $\geq 90\%$ of the total seismic mass in each principal horizontal direction. Fig 19.
- The 65% Rule (Irregular Modes Check): The cumulative value of the first three lateral translational modes must be at least 65% in each plan direction. See Fig 18.

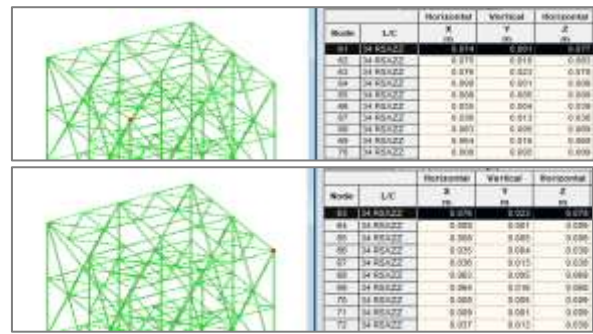


Fig-16: Specimen 1 - Displacements at critical joints (red dots) (0, 6, 30) & (6, 6, 30) for torsion check (STAAD.Pro extract)

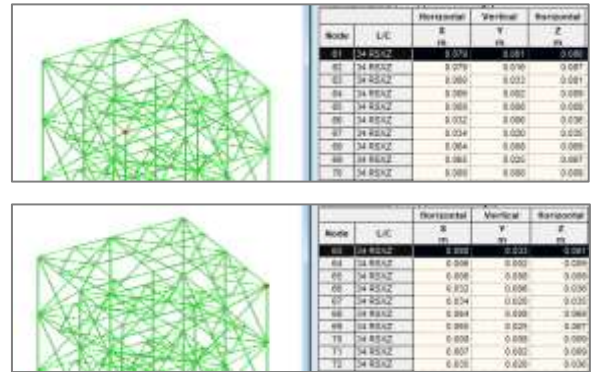


Fig-17: Specimen 2 - Displacements at critical joints (red dots) (0, 6, 30) & (6, 6, 30) for torsion check (STAAD.Pro extract)

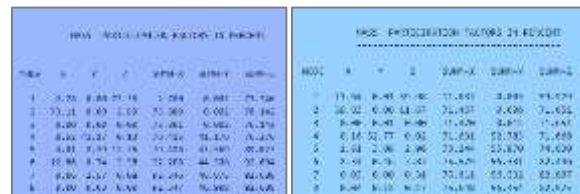


Fig-18: Modal Mass Participation 65% rule - Specimen 1(left) & Specimen 2 (right) (STAAD.Pro extract)

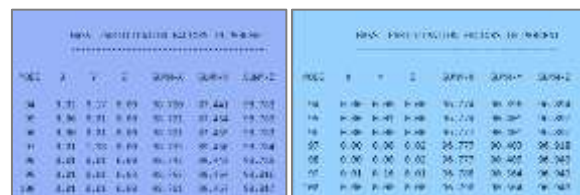


Fig-19: Modal Mass Participation 90% rule - Specimen 1(left) & Specimen 2 (right) (STAAD.Pro extract)

D. Post Analysis Observations from Phase 2

Once the pushover analysis is performed for an initial trial push (0.45 considered), during the post processing stage, the value is reset to inelastic target displacement which is otherwise called as performance point for evaluation. Also, the model behaves differently based on the type of vertical load distribution initiated. Both has significant effect on the Demand - Capacity curve developed. The below sections explain these aspects.

(1) Calculated Target Displacement: Performance Point (FEMA-356)

Since MRSA uses elastic response reduction factor ($R=4.5$, for SCBF), the real inelastic seismic drift is underrepresented. Therefore, the real inelastic displacement demand (d_i) known as performance

point is obtained using the criteria of IS 1893 (Part 1): 2016 and FEMA 356 framework.

$$d_t = C_0 \times C_1 \times C_2 \times C_3 \times S_a \times (T_e/2\pi)^2 g \text{ -----(8)}$$

- C_0 (Modification Factor for SDOF to MDOF): For a 6-storey building w/ uniform load, $C_0=1.42$.
- C_1 (Inelastic Displacement Demand): For a structure with a short-to-medium period, $C_1=1.0$
- C_2 (Hysteresis Stiffness Degradation): For steel structures with bracing elements that experience minimal cyclic degradation, $C_2 = 1.0$
- C_3 (P-Delta Multiplier): = 0.9, from Table 6, strength degradation is negligible
- T_e (Effective Fundamental Period): The elastic period of the building determined from the structural stiffness in the direction under consideration.
- S_a (Spectral Acceleration): relates to T_e and damping ratio of the structure. g (Acceleration due to gravity): 9.81m/s^2
- The final recalculated d_t is shown in the DCR curves for the specimen, refer to Fig 20 & 21.

(2) Specimen Behaviour for Methods 1& 3 Load Distribution

The Table 5 summarizes the structural behaviour for methods 1 and 3 and lists relative merits and demerits. One limitation here is that the combined envelope of the maximum response from both methods is not possible to incorporate in the analysis command line.

(3) Demand-Capacity Curves for the Specimens

Fig 20 & 21 shows DCR curves for specimens 1& 2 respectively, extracted from STAAD.Pro Pushover analysis of the specimens. The individual components in the graph are:

- Demand Curve: Represents the seismic hazard/earthquake energy (often expressed as a response spectrum).
- Capacity Curve: Represents the building's structural lateral resistance (base shear vs. roof displacement).
- Both curves are overlapping in the graphs below, this signifies much elastic strength and conservative design. The point where the demand curve intersects the capacity curve is the Performance Point. It is shown by hidden lines intersecting in the graphs below. Both specimens remain linear elastic up to this point.

Table 5: Structural Behaviour for Load Distribution Methods 1 & 3

Sl No:	Behavioral Indicator	Method 1 (Parabolic / (k=2))	Best-fit (Method 1)	Method 3 (Uniform/ Mass Only)	Best-fit (Method 3)
1	Global Profile Shape	Flexural Cantilever Curve, overturning action	¹ 2XB	Rigid Body Uniform Shear, sliding forward action	¹ 1XB
2	Primary Stress Mechanism	Base Overturning Moment	¹ 2XB	Storey Shear Force Accumulation	¹ 1XB
3	Hinge formation sequence	Floors 5 & 6 reach the tension yielding and	¹ 1XB	Floors 1 & 2 fail rapidly in compression buckling.	¹ 2XB

		compression buckling limits first.		Foundation-level brace buckles first	
4	Capacity Curve Yield	Lower ultimate base shear at performance point	¹ 1XB	Higher ultimate base shear at performance point	¹ 2XB

Tag Note 1: Method 1 is dominated by flexure and overturning whereas Method 3 is dominated by shear. 1XB excels in method 3 because it is less slender and has more rigid box like framework of dedicated tension and compression member fighting the story shear at each storey. For 2XB, even if slenderer, as height increases, the diamond framework of 2XB becomes more efficient in resisting overturning [28] and consequently P-delta effect will be less as seen in Table 6 results.

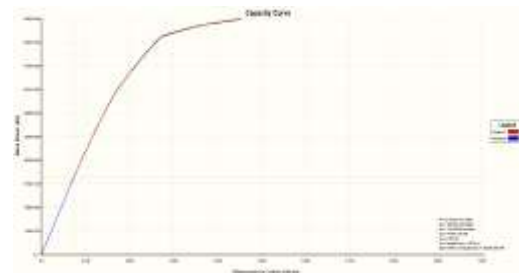


Fig-20: Demand-Capacity Curve - Specimen 1 STAAD.Pro extract)

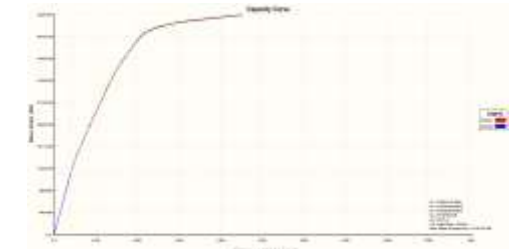


Fig-21: Demand-Capacity Curve - Specimen 2 STAAD.Pro extract)

E. Seismic Evaluation of the Specimens

The seismic evaluation of the specimens is performed based on Key Performance Indicators (KPIs) as tabulated below in Table 2. These KPIs cover the outcomes of both the seismic analysis and evaluation phases.

(1) FEMA-356 Acceptance Criteria – Overview and selection

Performance evaluation level is chosen as Life Safety (LS). In this evaluation level, while major structural and nonstructural components are damaged, the primary goal is to prevent fatalities by ensuring the building maintains its gravity-load-carrying capacity and exits remain clear. This will safeguard the refinery machinery and equipment along with occupants. In this classification, the steel components are classified:

- Deformation controlled: Braces in Tension
- Force Controlled: Braces in Compression, Beams and Columns

(2) System Level Checks – for Life Safety (LS) from FEMA-356

Table 6 below tabulates the system level life safety checks.

Table 6: System Level Acceptance Criteria – FEMA-356 (LS)

SI No:	Action	Limit s	Acceptance limits		Remarks
			Specimen 1	Specimen 2	
1	Inter storey Drift	≤ 76 mm 1.5% Drift	76:120 (at roof) - safe	80:120 (at roof) - safe	120mm max
2	P-Δ Strength degradation stability	$10 \leq 0.10$ ideal	0.009	0.006	okay
3	Plastic Hinges	By Count at d_t	2(IO) green hinges – okay for LS	Nil	To be on the left of yellow hinge in model/LS

Tag Note 1(θ): Stability Coefficient $\theta = \frac{P\Delta}{Vh}$; $\theta \leq 0.10$ ideal, $0.10 < \theta \leq 0.25$ significant

➤ θ for specimen1 = $1368 \times 6 \text{ kN (Seismic weight)} \times 0.45 / (13103 \times 30) = 0.009$

➤ θ for specimen1 = $1368 \times 6 \text{ kN (Seismic weight)} \times 0.45 / (21206 \times 30) = 0.006$

V=base shear, P = seismic weight, h=height, Δ = push dimension at V taken

3) Component Level Checks – for Life Safety (LS) from FEMA-356

At performance point for the structure the following component level life safety checks are performed for the structural elements as tabulated in Table 7. These are evaluated mainly based on the acceptance level hinges formed rather than measuring the drift occurred as shown below.

Table 7: Component Level Acceptance Criteria – FEMA-356 (LS)

SI No:	Action		Limits (measured at d_t)	Acceptance limits		Relative merit
				Specimen 1	Specimen 2	
1	Deformation controlled axial action	Brace in Tension	$\Delta \leq 6.0\Delta_y$	2 (IO) hinges at d_t	No hinge at d_t	¹ 1XB
2	Deformation controlled axial action	Brace in Compression	$\Delta \leq 4.0\Delta_y$	2 (IO) hinges at d_t	No hinge at d_t	¹ 1XB
4	Force controlled combined axial & flexure action	Columns (to remain fully elastic)	if P/Pu ≥ 0.5 , Demand to Capacity ≤ 1.0	2 (IO) hinges at d_t , safe for LS	No hinge at d_t , hence okay	¹ 1XB
4	Deformation controlled axial action	Beams (FEMA 356 Table 5-5)	Rotation Limits: $\theta = 1.0\theta_y$	2 (IO) hinges at d_t	No hinge at d_t	¹ 1XB

Tag Note 1(¹1XB): While 2XB is in safe limits with heavier sections, 1XB needs only lighter sections. Δ_y is yield displacement and Δ is the displacement at d_t , Δ and θ are assessed through hinges formation.

(4) KPIs for the Specimens

The KPIs for the specimens are formulated based on the post analysis checks, post analysis observations from phases 1 & 2 and also based on the FEMA-356 acceptance criteria for Life Safety level in linear dynamic procedure category. Although FEMA-356 acceptance criteria are mainly used for seismic retrofitting, here it is used as a tool for understanding how the two LLRS respond to the dynamic analysis. These are tabulated in Table 8.

Also, the behaviour of the overall geometry sometimes exceeds the micro analysis results. Such as diamond pattern of the two-storey split X brace versus the box like rigid pattern of standard X brace. Two- storey spit X pattern is very effective as the storey height increases but standard X pattern is effective on moderate stout geometry. It is observed from table 8 that in 30m height this effect comes into action.

The tag notes

Table 8: KPIs for the Specimens

SI No:	Description	Outcome		Best-fit
		Specimen1 (2XB)	Specimen2 (1XB)	
1	Handling Overall Building Drift - Phase 1	76mm	80mm	¹ 2XB
2	Handling Base Shear – Phase 1	17691kN (X direction)	17548kN (X direction)	² 2XB
3	Handling Overturning Moment at Base - Phase 1	445449 (Mx)	470943(Mx)	³ 1XB
4	Handling P-Delta effect and overall stiffness - Phase 1	Fig 16	Fig 17	⁴ 2XB
5	Handling Torsional Resistance - Phase 1	Fig 16	Fig 17	⁵ 1XB
6	Optimized Member Design Efficiency- Phase 1	Fig 16	Fig 17	⁶ 1XB
7	Efficiency in DCR Curve Outcome for LS - Phase 2	More plastic hinges & base shear	Less plastic hinges & less base shear	⁷ 1XB
8	Efficiency in Component Level Checks for LS- Phase 2	Safe w/heavier sections	Safe w/lighter sections	⁸ 1XB
9	Efficiency in System Level Checks for LS- Phase 2	Safe w/heavier sections	Safe w/lighter sections	⁹ 1XB
10	Efficiency in Base Shear at d_t (DCR) - Phase 2	21206kN	13104kN	¹⁰ 2XB
11	Efficiency in Overturning Moment (DCR) - Phase 2	64497kN base shear@ 0.45m push	45279kN base shear@ 0.45. push	¹¹ 2XB
12	No of Plastic Hinges formed at d_t - Phase 2	2 (IO)	Nil	¹² 1XB
13	Suitability for midrise industrial structures – aspect ratio 2.5	Brace angle=59.0 4°	Brace angle=39.81°	¹³ 1XB
14	Suitability for fabrication ease	Less detailing for collector	More detailing for collector	¹⁴ 2XB

denoted thus ‘^x1XB’ & ‘^x2XB’ in the KPIs table 8 are explained below.

Note 1: The 2-storey X-brace creates a more continuous, truss-like mega-diamond pattern over the height of the structure as height increases. This distributes overturning moment more uniformly and helps in reducing overall drift at roof level.

Note 2: For base shear 1XB is supposed to be superior in efficiency. However, here for 30m tall structure, 3 levels of two-storey split X-brace diamond configuration are present. Its combined flexural bending resistance is very strong when compared to the resistance given by rigid box like standard X-brace configuration.

Note 3: Mode 1 Mx value in kN, higher value is more efficient.

Note 4: As seen in Fig 16 and 17, lesser global drift indicates higher resistance to P-delta effect.

Note 5: As seen in Fig 5.2 and 5.3, torsional resistance check will yield more value for 2XB. This is evident from the fact that 1XB is a stiffer LLRS.

Note 6: As seen from the member take off for both specimens, although more in quantity, 1XB optimizes for lesser sizes which when compared to 2XB gives less

quantity by weight. This is because 2XB is slenderer than 1XB. Hence, more efficient.

Note 7: For 2XB, more plastic hinges are not welcome although base shear is higher.

Notes 8&9: The component level and system level checks are pass for both 1XB & 2XB but member sizes are heavier for 2XB

Note 10: As seen in the DCR curves for both 2XB and 1XB, base shear is more for 2XB configuration owing to the fact that it is more ductile widely spread compared to 1XB and its diamond configuration works efficiently.

Note 11: Base shear directly relates to overturning moment efficiency

Note 12: From the DCR curve, two data points are seen within the target displacement for 2XB whereas no data points seen for 1XB

Note 13: Brace angle θ is steeper for 2XB owing to the fact that 2XB splits the single storey X brace across two stories. $\theta = \tan^{-1} 10/6 = 59.04$ for 2XB; near to upper limit (30-60)

$\theta = \tan^{-1} 5/6 = 39.81$ for 1XB; ideal, however, if a passage comes in the line of brace, less access available at 1XB, so 1XB is best fit here.

Note 14: Since the 1XB braces are directly connected to beam ends for both upper and lower-level braces, 4 gussets per beam have to be detailed and it is crowded at the joint. Gusset detailing required only for the middle beam at mid-point for 2XB, where it crosses.

(5). Final Verdict on Optimal LLRS

From the analysis results, observations, FEMA-356 checks and KPIs for the specific idealized refinery steel specimens, specimen 2 which is having standard X-brace geometry shown in Fig 6 is structurally superior and more cost effective. Although, both specimens exhibit equal KPIs, the tilt in favor of 1XB is owing to the KPIs no:13 where if more access to pipe line and egress required, 2XB is the better choice.

Final Verdict:

1XB got 8 best fit points

2XB got 6 best fit points

Standard X-Brace is the optimal LLRS for the specimen

VI. DISCUSSION OF RESULTS

The relative merits of Standard X-brace and Two-Storey Split X-brace are described below although Standard X-brace has significant advantages among the two configurations as evident from the previous sections.

A. Relative merits of Standard X-brace LLRS

The bracing members cross at a central node that does not inherently place a heavy vertical point load on the horizontal floor beam. Since the tension and compression actions almost balance out at the framing joints, the collectors are sized purely for gravity and axial drag forces.

Both these aspects make lighter and more economical collectors.

Unlike a chevron brace, where when the compression brace buckles during an earthquake, massive downward or upward unbalanced force is created, standard X-brace keeps a balanced composition which avoids this post buckling imbalance [28]. Therefore, the beam need not be designed for localized plastic bending capacity [28].

Since the load paths are straightforward for Standard X-brace, rigorous overstrength calculations are simplified and hence simplicity in design checks prevails.

B. Relative merits of Two Storey Split X-brace LLRS

In standard X-bracing, four major elements such as column, beam and two massive diagonal braces (upper and lower) and its gusset plates all converge at a single beam – column intersection. This always causes crowding of welding and bolting. Split X-brace moves the brace intersection to the middle of the beam, thereby shifting half of the gusset plate detailing away from the critical beam to column zone. Therefore, two storey split X-brace is clear advantage when it comes to eliminating connection congestion.

Since the braces split over the height and width of a two-storey setup, the node-to-node length of a single diagonal is shorter than that of single full bay X-brace [28]. Eventually, this makes it easier to pass the code prescribed slenderness check as mandated in IS18168:2023 Table 2 for fuse elements.

Ease of fabrication is the most significant advantage of the two-storey split x-brace configuration. As understood from the above two aspects, end connections will be simpler and not at all crowding. Moreover, architectural appeal is always with two storey split X-brace configuration and it provides better pipe clearance and even better egress for a refinery structure.

End of the day, more economical and lighter material will be the clear choice unless it is specifically decided the other way.

VII. CONCLUSION

Selecting LLRS for a structure is a discreet choice of the structural engineer after evaluating various factors. Seismic considerations ensuring the capacity of the LLRS meeting or exceeding the seismic demands understood through seismic analysis is the top priority. Specific to each structure, the seismic demand vs capacity varies and it must be thoroughly analyzed with respect to the seismic parameters and code provisions. The crux of the analysis is finding an optimal design that satisfies all required dynamic responses [29]. Architectural constraints towards a particular type of LLRS may come implicitly from the client or owner. Structural engineers often adjust designs to meet or exceed code requirements, typically balancing structural integrity against architectural constraints. In industrial refinery steel construction environment, the refinery process flow directly mediates this tradeoff.

The structural analysis and design landscape is shifting dynamically [29, 30], with the findings in the analysis chapter highlighting the transition from code prescribed

Capacity-Based Design (CBD) approach to modern Performance-Based Design (PBD). Nonlinear static or dynamic analysis procedures are essential to PBD approach [31]. In this approach, the seismic safety of the structure is given paramount importance. The designer goes beyond the conventional code-based instructions to analyze the structure based on seismic structural performance levels categorized by FEMA-356 for which the building is guaranteed to the client [27].

However, code compliance is vital for the analysis. For example, the code [3] prescribes maximum 0.4% drift limit (roof level displacement) which is an essential criterion for the analysis of LLRS. To take care of accidental torsion, the code [3] prescribes 5% increase in moment on each side of the center of mass while performing seismic analysis. The code prescribed slenderness limit and yield strength [5] forms the criteria upon which demand versus capacity is assessed.

Based on the discussion above, this dissertation hopes to offer modest insights that may assist future researchers.

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