Pre-Engineered Buildings with Crane: Designed by Indian and American Code

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doi.org/10.64643/IJIRTV13I1-204496-459

***Abstract*—Over the past few decades, technological advances have significantly improved the quality of human life. Pre-engineered buildings (PEBs) represent one such advancement within the construction industry. Although the concept originated in the 1960s, its widespread application has increased only in recent years. PEBs introduce a modern approach to constructing single-story industrial structures, emphasizing the selection of optimal structural components based on functional requirements. These buildings are primarily developed through pre-design and prefabrication techniques. The growth of PEB technology is driven by advantages such as reduced construction time, cost efficiency, enhanced structural performance, and improved architectural flexibility. In this study, a pre-engineered building is designed with multiple parameters using the STAAD-Pro software and analyzed under various loads, including dead load, live load, collateral load, wind load, and their appropriate combinations. The main objective is to understand the fundamental principles of PEBs and to determine the minimum structural weight while ensuring acceptable displacements and force distributions that contribute to safety and stability. This work also presents a detailed design of a PEB incorporating crane movement, evaluated using both Indian and American standard codes.**

***Index Terms*—Pre-engineered buildings, Industrial building with crane structure, STAAD-Pro.**

I. INTRODUCTION

This study aims to address the key challenges associated with Pre-Engineered Building (PEB) design by evaluating the underlying design philosophies, fabrication practices, and erection techniques prescribed in both Indian and American code systems. Through this comparative approach, the study seeks to ensure that the resulting structural solution is not only safe and cost-effective but also well-suited to industrial applications where integrated crane operations play a critical role in day-to-day functionality.

Pre-Engineered Buildings (PEBs) are an efficient structural system widely used for non-residential and industrial applications. Developed initially in the United States, PEBs are fabricated using standardized steel components and assembled on-site, enabling rapid construction and reliable quality control. Their design optimizes material usage by aligning structural geometry with bending moments, resulting in lighter and more economical structures. Capable of achieving large column-free spans and considerable heights, PEBs are suitable for warehouses, factories, power plants, and commercial facilities. In addition to structural efficiency, PEBs offer reduced construction time, lower lifecycle costs, minimal maintenance requirements, and improved durability, making them a sustainable alternative to conventional construction methods.

Pre-Engineered Buildings (PEBs) have become a alternative to conventional steel structures due to their cost efficiency and rapid construction timelines. However, when these buildings are intended for industrial use, particularly those incorporating crane operations, the design process demands careful attention to multiple factors, including load combinations, structural optimization, and adherence to international standards.

One of the key challenges is comparing Indian Standards (IS 800:2007, IS 875) with American codes such as ASCE, AISC, and IBC to identify the most effective design approach in terms of structural weight, cost, and safety. Achieving this requires optimizing member sizes based on bending moment distribution and validating structural performance through advanced analysis tools like STAAD-Pro. The goal is to design the structure with minimum steel usage with ensuring strength, stability, and compliance with safety norms.

II. LITERATURE REVIEW

Lovneesh Sharma, Nileshwar Taak, and Pankaj Kumar Mishra (2021) [1], This study compares PEBs and CSBs across multiple performance parameters. Using Bentley STAAD-Pro, two structurally identical buildings were designed—one as a PEB and the other as a CSB. The analysis revealed that PEBs offer significant advantages in construction speed, cost efficiency, and material optimization. Reduced Dead load and better material utilization make PEBs a sustainable and economical alternative to conventional steel systems.

Shaik Kalesha, B.S.S. Ratnamala Reddy, Durga Chaitanya Kumar Jagarapu [2], the authors emphasize that PEB systems employ prefabricated steel components manufactured in controlled environments, ensuring uniform quality and reducing on-site labor. PEB construction can be completed in 6–8 weeks, much faster than CSBs. Steel, being infinitely recyclable, aligns with sustainability goals. The study found that PEB structures weigh about 50% less than CSBs, resulting in approximately 35% cost savings.

G. Sai Kiran, A. Kailasa Rao, R. Pradeep Kumar [3], this research highlights that PEBs designed under modern codes such as IS 800:2007 or MBMA are lighter and more economical than those designed under older standards like IS 800:1984. The study also examined seismic and wind load considerations for structures in Zone IV with wind speeds of 47 m/s. It concluded that international codes like AISC/MBMA are often preferred due to their provisions for slender sections, which IS 800:2007 does not address.

Aakash Varma and Dr. Rajeev Chandak [4], the authors analyzed PEB behavior under various loads using STAAD-Pro V8i. They highlighted the efficiency of tapered I-sections, which align better with bending moment diagrams, improving material utilization. PEB systems allow modular design and mass production, reducing construction time and costs while promoting sustainability.

Shubham Dashore and Shraddha Kanojiya [5], this study designed a PEB warehouse considering dead, live, wind, and seismic loads using STAAD-Pro and Indian codes IS 800:2007 and IS 1893. Findings indicate that reduced structural mass and effective bracing systems enhance seismic performance. Cold-formed sections designed by IS 801 and AISI standards further improve resilience.

M. Varshitha and B D V Chandra Mohan Rao [6], the authors compared two steel buildings—one PEB and one CSB—and found that PEB required 24.9% less steel. Wind load combinations were more critical than seismic loads. PEBs demonstrated advantages such as reduced cost, faster construction, and better aesthetics, making them preferable for industrial applications.

Moiz Mohammed and Ahmed Hussain Mohammed [7], this research focused on designing an Airbus A-380 hangar using STAAD-Pro. The study revealed that bay spacing significantly influences steel quantity, with larger spacing reducing primary member weight but increasing secondary member weight. Lightweight members improve wind resistance and reduce foundation costs.

Animesh Tripathi, Rituraj, Shezad Meman, Nishant Patil [8], the authors compared PEB designs under IS 800:2007 and AISC 360-10. AISC-based designs yielded lighter sections due to lower imposed loads and serviceability criteria. Steel weight reductions of up to 27% were observed under AISC compared to IS codes, making AISC more economical for long-span structures.

Pradeep V and Papa Rao G [9], this study demonstrated that PEB frames are lighter and more flexible, offering better seismic resistance. Roof structures in PEBs were 26% lighter than CSBs, and foundations were simpler and less costly. Overall, PEBs were found to be 30% more economical than CSBs.

Pratik R. Atwal, Vinaysingh Chandrakar, and Pravinsingh Tomar [10], this work compares designs prepared under IS 800:2007 with those based on international standards for a school building, with loading parameters consistently taken from Indian codes. Analysis and design were conducted using STAAD-Pro and manual checks. The results show notable reductions in steel tonnage when international standards are applied: Euro-03 reduces weight by 27.2%, and BS 5950-2000 reduces weight by 9.04%, relative to IS 800:2007. The study discusses code-level differences including bracing slenderness limits (≤ 120), limit state design alignment with Eurocodes, and the continued presence of ASD provisions within IS 800:2007 highlighting IS 800's hybrid nature and Indian practice-oriented safety factors.

III. METHODOLOGY

This paper focuses on the structural design and analysis of an industrial building proposed to be situated in Pune, Maharashtra. The facility has been conceptualized as a Pre-Engineered Building (PEB) with an overall length of 88 meters and a width of 29.2 meters, providing a clear internal height of 10 meters. The roof is designed with a gentle slope of 1:20 and is intended to support an additional solar panel load of 25 kg/m². The structure incorporates bay spacing of 8 meters along its length. The design and analysis have been performed in compliance with the relevant provisions of Indian Standard codes as well as American Standard specifications. STAAD-Pro software has been used as the primary tool for structural modelling, load application, and member design. The analytical model includes detailed geometric features, member properties, connectivity between nodes, and necessary eccentricities to realistically simulate the building's behavior.

Location	Pune
Length	88 m
Width	29.2 m
Clear Height	10 m
Bay spacing	11 bays -8.0 m bay spacing
Basic wind speed	39 m/sec
Seismic zone	III
Cyclonic region	No
Span	1@29.2m
Crane data	15MT.

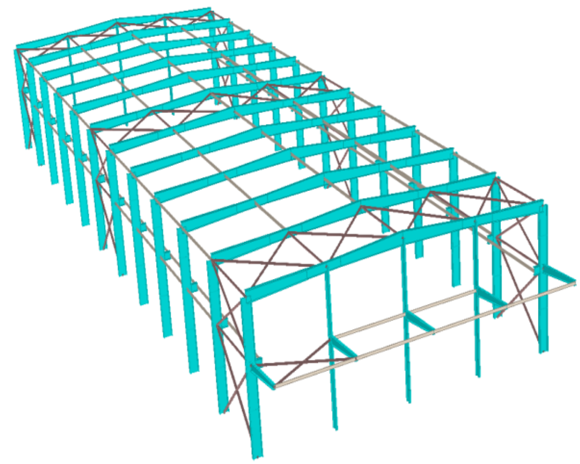

Fig. 3.1 Isomeric 3D rendered view of PEB

IV. DESIGN AND ANALYSIS

Load Calculation:
1.Dead Load:
Load determination in STAAD-Pro, self-weight is calculated automatically using the assigned density and sectional properties of steel members. IS 875 (Part 1) forms the basis for dead load evaluation in Indian practice. ASCE 7 16 (Chapter 3) provides similar guidelines in the American code. Additional roof related components are introduced manually as distributed loads.

Roof Dead is inclusive of following,
Standing seam Panels 18" 0.55mm Thick = 5.50 Kg/Sqm, Liner Panel = 4.50 Kg/Sqm, Insulation-50mm Thk.,48 Kg-Cum = 2.40 Kg/Sqm, Standing Seam Accessories = 1.60 Kg/Sqm, Purlins = 6.00 Kg/Sqm, Total Dead Load on Roof = 20.00 Kg/Sqm.

Wall Dead load is inclusive of following,
Wall Panels 0.5mm Thick = 4.50 Kg/Sqm, Girt = 5.50 Kg/Sqm, Accessories = 2.00 Kg/Sqm, Total Dead Load on wall = 12.00Kg/Sqm
Dead load on intermediate frame rafter = 0.20 x 8 = 1.6 kN/sqm.
Dead load on 1st & 12th frame rafter = 0.20 x 4 = 0.8 kN/sqm.
Dead load on intermediate frame column = 0.12 x 8 = 0.96 kN/sqm.
Dead load on 1st & 12th frame column = 0.12 x 4 = 0.48 kN/sqm.

2. Live load:
Indian Standards:
IS 875 (Part 2): According to the code, live load on the roof depends on the roof's configuration and usage. According to IS: 875 (Part 2) - for roof with no access provided, the live load can be taken as 75 kg/m2.
Live load on intermediate frame rafter = 0.75 x 8 = 6.0 kN/sqm.
Live load on 1st & 12th frame raft after = 0.75 x 4 = 3.0 kN/sqm.

American Standards:
ASCE 7-16 provides a structured and rational approach to determining roof live loads by considering the likelihood of uniform loading across different roof areas. The standard load of 0.96 kN/m² may be reduced for large tributary areas down to a lower limit of 0.58 kN/m², ensuring realistic load estimations while maintaining structural safety.
Live load on intermediate frame rafter = 0.58 x 8 = 4.64 kN/sqm.
Live load on 1st & 12th frame rafter = 0.58 x 4 = 2.32 kN/sqm.

3. Collateral load calculation:
Collateral Load- 50 kg/sqm on entire Roof building. This load includes utility load, solar panel load of 25 kg/sqm, and lighting load.
Collateral load on intermediate frame rafter = 0.50 x 8 = 4.0 kN/sqm.
Collateral load on 1st & 12th frame rafter = 0.50 x 4 = 2.0 kN/sqm.

4. Seismic load calculation:
IS 1893 (Part 1) – Indian Seismic
As per IS1893-(PART I)-2016 seismic zone for Pune is Zone III
Earthquake Zone Factor Z =0.16,
Importance Factor I = 1,
Damping Ratio D =5%,
Response Reduction Factor R = 4,
Soil Type ss = Medium soil,
h=11.23m

Fundamental natural translation period Ta for steel MRF building as calculated below,
$Ta = 0.085 X h^{0.75}$
$Ta = 0.085 X (11.23)^{0.75}$
Ta = 0.5214 s.
Soil type, Sa/g = 2.5, as per Cl. 6.4 of IS 1893 part 1,
Ah= (Z/2) X (Sa/g) / (R/I)
Ah = 0.05
ASCE 7 16 (Chapters 11–14) – American Seismic:
Occupancy Category = II (Medium Hazard)
Site Class = D
Mapped Spectral Response Short (Ss) = 0.33
Mapped Spectral Response 1 Second (S1) = 0.16
Seismic Importance Factor (I) = 1.00

Ordinary steel moment frames
Height Limit = 11.23 m
Site Coefficient (Fa) = 1.54
Site Coefficient (Fv) = 2.16
Maximum Spectral Response Short (SMS=Fa x SS) = 0.51
Maximum Spectral Response 1 second (SM1=Fv x S1) = 0.35
Design Spectral Response Short (SDS=2 / 3 x SMS) = 0.34
Design Spectral Response 1 second (SD1=2 / 3 x SM1) = 0.23
Seismic Design Category = C
Seismic Design Category = D
Basic Seismic Force Resisting System = Main Frames
Response Modification Factor (R) = 3.5
System Overstrength Factor (W0) = 2.5
Deflection Amplification Factor (Cd) = 3
Approximate period parameters (Ct) = 0.0724
Approximate period parameters (x) = 0.8000
Approximate Fundamental Period (Ta1) = 0.5012 Sec
Approximate Fundamental Period (Ta2) = 0.1000 Sec
Seismic Response Coefficient (Cs=SDS I / R) = 0.097
Structure Type = Sidewalls
Response Modification Factor (R) =3.25
System Overstrength Factor (W0) = 2
Deflection Amplification Factor (Cd) = 3.25
Approximate period parameters (Ct) = 0.0731
Approximate period parameters (x) =0.7500
Approximate Fundamental Period (Ta1) = 0.4484 Sec
Approximate Fundamental Period (Ta2) = 0.1000 Sec
Seismic Response Coefficient (Cs= SDS I / R) = 0.104

5. Crane load calculation:
Crane Data:
1. Crane Capacity = 15.00 MT
2. Bridge Span 30.00 M.
3. Bridge Weight = 184.3 KN
4. Trolly Weight = 7.90 KN

5. Wheelbase = 4500 MM
6. Number of Wheels = 2
7. Bay Spacing = 8000 MM

Reaction of crane as per Indian code:
Maximum Wheel Load (WL_{max})
WL_{max} = (Crane Capacity + Hoist Weight +0.5Bridge Weight)/Numbers of Wheels = (150 + 7.9 + 0.5 x 184.3)/2
WL_{max} = 125KN. (W/O IMPACT)
WL_{max} = 157KN. (WITH IMPACT)

Minimum Wheel Load (WL_{min})
WL_{min}=(10%(Crane Capacity+Hoist Weight) +0.5Bridge)/Nu. of Wheels = (10% (150 + 7.9) + 0.5 x 184.3)/2
WL_{min} = 54KN. (W/O IMPACT)
WL_{min} = 68KN. (WITH IMPACT)

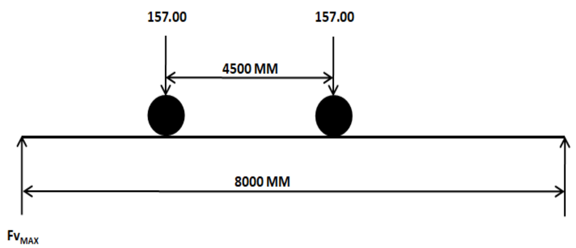

Fig. 4.15 Crane Load application A

Fv_{Max} =WL_{Max} [1+ (BAY SPACING-WHEEL BASE)/BAY SPACING] + RAIL WT. = 157 [1+ (8-4.5)8] +8 = 238 KN
Fv_{Max} =WL_{Min} [1+ (BAY SPACING-WHEEL BASE)/BAY SPACING] + RAIL WT. = 68 [1+ (8-4.5)8] +8 = 114 KN
Horizontal Thrust (Fh) = 10% (Crane Capacity + Hoist Weight) = 15.51KN.
Traction Force (F_t) = 5% of Static Wheel Load = 8KN

Reaction of crane as per American code:
Maximum Wheel Load (WL_{max})
WL_{max} = (Crane Capacity + Hoist Weight +0.5Bridge Weight)/Numbers of Wheels = (150 + 7.9 + 0.5 x 184.3)/2
WL_{max} = 125KN. (W/O IMPACT)
WL_{max} = 138KN. (WITH IMPACT)

Minimum Wheel Load (WL_{min})
WL_{min} = (10% (Crane Capacity + Hoist Weight) +0.5Bridge)/Nos of Wheels = (10% (150 + 7.9) + 0.5 x 184.3)/2
WL_{min} = 54KN. (W/O IMPACT)
WL_{min} = 60KN. (WITH IMPACT)

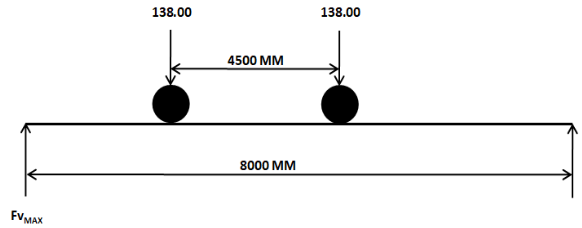

Fig. 4.16 Crane Load application B

Fv_{Max} =WL_{Max} [1+ (BAY SPACING-WHEEL BASE)/BAY SPACING] + RAIL WT. = 138 [1+ (8-4.5)8] +8 = 208 KN
Fv_{Max} =WL_{Min} [1+ (BAY SPACING-WHEEL BASE)/BAY SPACING] + RAIL WT. = 60 [1+ (8-4.5)8] +8 = 94 KN
Horizontal Thrust (Fh) = 10% (Crane Capacity + Hoist Weight)
= 15.51KN.
Traction Force (F_t) = 5% of Static Wheel Load = 6.90KN

6. Wind Load (WL):
In structural engineering, the horizontal wind load acting on a structure must be incorporated into the design of buildings, towers, and similar systems. The magnitude of this aerodynamic force is governed by wind velocity characteristics, the aerodynamic geometry of the structure, its exposed surface area, and its geographical and topographical positioning relative to prevailing wind patterns.
IS 875 (Part 3) – Indian Standard:
Job Site Location = Pune, Maharashtra.
Basic Wind Speed (Vb) = 39 m/s.
Probability Factor (k1) = 1 (For general buildings & 50years return period -Table 1)
roughness & height factor k2 = 1.07 (Terrain Category 1-Table2)
Topography Factor (k3) = 1
Importance factor for cyclonic region (k4) = 1 (site location away from Arabian sea coast)
Design wind speed (Vz) = Vb x k1 x k2 x k3 x k4
Vz = 39 x 1 x 1.07 x 1 x 1
Vz = 41.73 m/s
Wind pressure (Pz) = 0.6 x Vz2 (IS875-2015-CL 6.3)
Pz = 0.6 x 41.432 /1000
Pz = 1.045 KN/Sqm
Design Wind Pressure (Pd) = Pz x kd x ka x kc

Wind Directionality factor (kd) = 0.9
Area Averaging Factor (ka) = 0.816
Column Tributary = 8 x 11 = 88sqm <100sqm
Column Tributary = 8 x 11 = 88sqm <100sqm
Ka lies between 0.8 to 0.9.
Combination factor (kc) = 0.9
Design wind pressure for Frames (Pd) = 0.732 KN/Sqm
Internal Pressure Coefficient (Cpi) = +/- 0.5
External Pressure Coefficient (Cpe)

Table 4.1 External Pressure Coefficient

Parameter		Angle	EF	GH	EG	FH
h/w= 10.8/29.2	0.37	0	-0.8	-0.4	-0.8	-0.4
l/w = 88/29.2	2.97	5	-0.9	-0.4	-0.8	-0.4
Cpe		2.863	-0.86	-0.4	-0.8	-0.4

Table 4.2 Summery of wind load

Span = 8.00	A	R1	R2	B	C	D
WLLP	1.17	-7.96	-5.26	-4.38	-5.87	-5.87
WLRP	-4.39	-5.27	-7.95	1.17	-5.87	-5.87
WLLS	7.02	-2.11	0.585	1.46	-0.53	-0.53
WLRS	1.46	0.58	-2.10	7.02	-0.53	-0.53
WLE1P	-5.85	-7.61	-7.60	-5.85	1.06	-3.20
WLE1S	0.00	-1.75	-1.75	0.00	6.40	2.13
WLE2P	-5.85	-7.61	-7.60	-5.85	-3.20	1.06
WLE2S	0.00	-1.75	-1.75	0.00	2.13	6.40
Span = 4.00	A	R1	R2	B	C	D
WLLP	0.59	-3.98	-2.63	-2.19	-5.87	-5.87
WLRP	-2.19	-2.63	-3.98	0.59	-5.87	-5.87
WLLS	3.51	-1.05	0.29	0.73	-0.53	-0.53
WLRS	0.73	0.29	-1.05	3.51	-0.53	-0.53
WLE1P	-2.93	-3.80	-3.80	-2.93	1.07	-3.20
WLE1S	0.00	-0.88	-0.88	0.00	6.41	2.14
WLE2P	-2.93	-3.80	-3.80	-2.93	-3.20	1.07
WLE2S	0.00	-0.88	-0.88	0.00	2.14	6.41

ASCE 7 16 (Chapters 26–30) – American Standard:
Job Site Location = Pune, Maharashtra.
Basic Wind Speed (V_b) = 39 m/s.
ASCE7-2016 Wind Speed Conversion & Rules:

ASCE7-2016 uses the same procedure as ASCE7-2010, but with return periods for Risk Category IV as follows:
Risk Category I — 300-years
Risk Category II — 700-years
Risk Category III — 1700-years
Risk Category IV — 3000-years
Wind Speed Conversion (50-year → MRI-based)

Wind speed for each return period is determined by multiplying the 50-years wind speed V_{50} by factor F as follows:

$V = [0.45 + 0.085 \ln(12T)] \times V_{50}$
$V = [0.45 + 0.085 \ln (12 \times 700)] \times 39$
V = 47.58 m/s
V = 171.288 km/h
Max Wind speed = 171.288 km/hr
Building Category = 2
Exposure Category = C
Exposure Coefficient (K_z) = 1.04
Topographic Factor ($K_{zt}*K_d$) = 0.85
Type of structure (Enclosed/Partially) = P
Wind pressure (q_h) $=0.613 \times K_z \times V^2 \times K_{zt} \times K_d$
$q_h = 0.613 \times 1.04 \times (47.58)^2 \times 0.85$
q_h = 1.23 KN/Sqm
Design wind pressure for Frames (q_h) = 1.23 KN/Sqm
Internal Pressure Coefficient (GC_{pi}) = 0.55 & -0.55

Table 4.3 Summary of Wind Load

Span = 8	A	R1	R2	B	C	D
WLLP	-1.47	-12.22	-9.05	-8.26	-8.97	-8.97
WLRP	-8.26	-9.05	-12.20	-1.47	-8.97	-8.97
WLLS	9.34	-1.378	1.771	2.558	0.898	0.898
WLRS	2.55	1.771	-1.378	9.348	0.898	0.898
WLE1P	-9.84	-12.22	-12.20	-9.84	-1.34	-7.54
WLE1S	0.98	-1.38	-1.378	0.984	8.530	2.335
WLE2P	-9.84	-12.22	-12.22	-9.84	-7.54	-1.34
WLE2S	0.98	-1.378	-1.378	0.984	2.335	8.530
Span = 4	A	R1	R2	B	C	D
WLLP	-0.73	-6.101	-4.526	-4.13	-4.49	-4.49
WLRP	-4.13	-4.52	-6.101	-0.73	-4.49	-4.49
WLLS	4.67	-0.68	0.886	1.279	0.449	0.449
WLRS	1.27	0.88	-0.689	4.674	0.449	0.449
WLE1P	-4.92	-6.10	-6.101	-4.92	-0.67	-3.77
WLE1S	0.49	-0.68	-0.689	0.492	4.265	1.167
WLE2P	-4.92	-6.11	-6.101	-4.92	-3.77	-0.67
WLE2S	0.49	-0.68	-0.689	0.492	1.167	4.265

V. RESULT AND DISCUSSION

A. Base shear comparison:

A comparative assessment (Fig. 5.1) reveals that the base shear values estimated using IBC 2018 are significantly higher than those derived from IS 1893. Quantitatively, the IBC-based base shear in the X-direction is approximately 1.8 times greater, while in the Z-direction it approaches nearly twice the values obtained from IS 1893. These discrepancies can be attributed to fundamental differences in seismic design philosophies, including response spectra definitions, seismic coefficient calculations, and safety considerations embedded within each code. The IBC framework adopts relatively conservative assumptions, leading to the prediction of higher seismic demands. Nevertheless, these increased forces are rationalized during the design stage through the application of suitable load combination factors, generally taken as unity. Conversely, IS 1893 yields comparatively lower base shear values, facilitating more economical designs, while incorporating a higher load factor (1.5) within its load combinations. Overall, the findings suggest that IS 1893 emphasizes design economy, whereas IBC 2018 prioritizes enhanced safety margins. Hence, the choice of design standard has a substantial impact on both the magnitude of seismic forces and the resulting structural design outcomes.

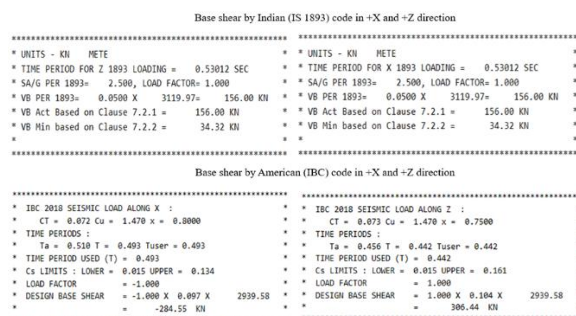

Fig. 5.1 Base shear comparison

B. Weight Comparison

Table 5.4 Weight Comparison

SR. NO.	Code	STAAD Weight	Unit
1	Indian Standard (IS800:2007)	82.5	(MT)
2	American Standard (AISC:2016)	70.3	(MT)
3	Reduction of weight	12.2	(MT)
4	% of weight Reduction	14.79	%

The results indicate that the AISC (LRFD)-based design yields a lower total steel weight compared to IS 800:2007, with a reduction of 12.2 MT (approximately 14.79%). This difference is mainly due to variations in design philosophy, where AISC promotes efficient material utilization and optimized member design, resulting in lighter sections. In contrast, IS 800:2007 adopts more conservative provisions, particularly in terms of serviceability and section classification, leading to heavier structures. The reduced steel usage in AISC design contributes to cost savings in fabrication, transportation, and erection. Overall, the AISC approach is found to be more economical, especially for pre-engineered and large-span industrial structures.

VI. CONCLUSION

The study reveals that designs based on AISC (LRFD) achieve nearly 14.79% lower steel consumption than those following IS 800:2007, under identical loading. This improvement is mainly due to comparatively lenient serviceability limits, reduced live load values, and more adaptable section classification in AISC. In contrast, IS 800 leads to increased member sizes because of stricter deflection control and conservative design provisions.

In terms of seismic response, IBC 2018 results in higher base shear compared to IS 1893, indicating a more conservative design philosophy. Although IS standards prioritize structural safety and reliability, they often result in higher material usage.

Overall, AISC provides a more economical and efficient approach for PEB design, while IS standards are preferable where stringent serviceability and higher safety margins are essential.

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