International Journal of
AI ELECTRICAL CIVIL AND MECHANICAL ENGINEERING
Peer Reviewed, Referred & Indexed Journal
ISSN: 3070-0434 www.zesterapublications.com Original Research Paper

STAAD-BASED ANALYSIS OF A SINGLE-STORY INDUSTRIAL STEEL SHED

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Abstract - Some of the most common types of buildings for industrial, logistical, manufacturing, and agro-processing purposes in India are steel industrial sheds with one story. An symmetrical, one-story industrial shed was analyzed in three dimensions using STAAD, and its compliance with the American Institute of Steel Construction (AISC) Allowable Stress Design (ASD) steel code was verified in this study.The SELECTseries 5 Pro V8i? With a roof pitch of around 19.6°, the structure has a clear span of 22.5 m, a total length of 42.0 m (seven bays at 6.0 m centre-to-center), eaves height of 4.5 m, and ridge height of 8.5 m. This space-frame model has 357 members and 191 joints. It has wide-flange columns (W14×43), web members and truss chords with angle sections (L80808/L60607), purlins with a pipe section (PIPS20), and bracing and eave struts with an angle section (L60606). By combining dead load, live load, and wind load from two directions into three different combinations, we were able to analyze the system as a linear elastic static space frame. Under a combined dead and wind load, the results reveal a maximum column bending moment of 14.16 kN·m and a maximum nodal displacement of 6.17 mm, both of which are well within the allowed range for serviceability. The AISC steel code check shows that the main columns and truss members are adequately designed (utilisation ratio 0.38-0.61). However, the PIPS20 roof purlins are severely overstressed (utilisation ratio up to 89.5) and the L60606 eave struts fail on slenderness (ratio 1.241), so it seems like they need to redesign that section. The article details a whole finite-element-based process for analyzing industrial sheds and shows how code-based verification is useful for finding insufficient member sections that could go unnoticed in handmade designs.

Important Terms - Steel structure for industrial use; STAAD.Space frame analysis, AISC ASD, wind load, truss, and purlin are all part of the pro package.

I. INTRODUCTION

Thanks to initiatives like Make in India and the development of new production corridors as well as rising need for logistics and storage facilities, India's industrial sector has grown substantially over the last 20 years. The one-story steel shed is the backbone of most industrial buildings, workshops, and warehouses due to its many advantageous structural features, such as its wide open spaces, ease of construction, minimal material requirements, and capacity to accommodate future growth. Industrial sheds often follow the same pattern as repeating portal-or truss-bent systems: X-bracing in the roof and wall planes resists lateral wind stresses, longitudinal purlins carry the roof cladding between neighboring frames, and steel columns support a triangulated roof truss that spans the building width. For the shed under consideration here, the four factors that influence the overall proportions are span, bay spacing, eaves height, and ridge height. The values for these parameters are 22.5 m, 6.0 m, 4.5 m, and 8.5 m, respectively.

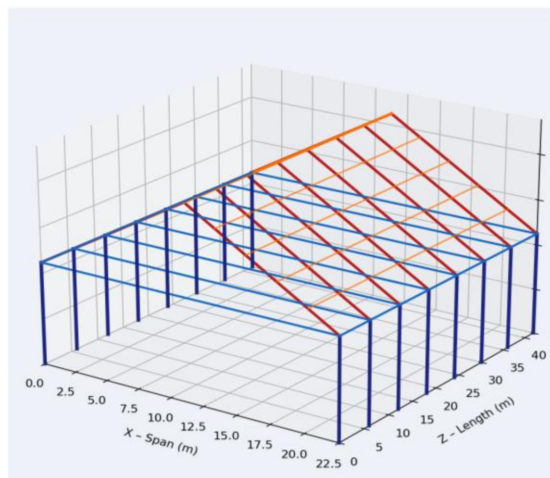

Fig. 1. A schematic in three dimensions of the industrial shed with one story, measuring 22.5 m × 42.0 m.

Manual truss and portal-frame approaches were the norm for shed design until the 1990s. These methods rely on simplifying assumptions and are not practicable for three-dimensional buildings that are uneven, have several bays, or are otherwise not perfectly straight. The stand.The most used structural analysis platform in Indian engineering practice is Pro V8i, produced by Bentley Systems. It uses the direct stiffness approach for space-frame analysis. In terms of shed design, it has features like automated self-weight calculation, handling of numerous load cases and combinations, Translational Repeat geometry creation, an integrated AISC/IS 800 steel code verification, and MEMBER TRUSS/MEMBER RELEASE commands for pin-jointed web members.

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A. Issue Description

The STAAD model was used to analyze this shed.Professional V8i (SELECTseries 5) in action. Here are the main project parameters retrieved from the STAAD: Table I.Pro file for input.

Parameter	Value	Unit
Building type	Single-storey industrial shed (steel)	—
Analysis software	STAAD.Pro V8i SS5	—
Total span	22.5	m
Total length	42.0	m
Bay spacing / no. of bays	6.0 / 7	m / —
Eaves height	4.5	m
Ridge height	8.5	m
Roof pitch	≈19.6° (1 in 2.81)	°
Total joints / members	191 / 357	—
Support nodes	26	—

Table I. Project Summary and Analytical Parameters

B. Objectives

- Create an exhaustive 3D stad.Detailed representation of the shed in space-frame format, including all relevant geometry, connections, sections, and boundary conditions.

- Determine the governing load combinations for linear elastic static analysis and apply them to dead, live, and wind load instances. Make sense of member forces, support responses, and nodal displacements by extracting and interpreting them.

- Verify the AISC ASD steel code and locate members that have been overstressed.

C. Scope

Seismic analysis and crane-girder design are seen as tasks for future development; hence, only wind and gravity loads are taken into account. There is no modeling of geometric and material nonlinearity or second-order P-Δ effects; the study is linear elastic. Design yield strength of 345 MPa is used for steel verification using AISC ASD; foundation and connection design are outside the scope of this study.

II. LITERATURE REVIEW

When dealing with determinate trusses, classical truss analysis with the method of joints and method of sections [20] provides precise answers. However, when dealing with members that convey bending, as is the case in actual shed frames, the results become approximate. The real three-dimensional reaction of a shed with movable purlins and non-symmetric wind coefficients cannot be captured by approximate lateral-load techniques like the Portal and Cantilever methods [25], [10], which were originally designed for manual analysis of building frames. Contemporary space-frame applications, such as STAAD, are based on the Direct Stiffness Method, which was first proposed by Turner et al. [21] and formally established by Przemieniecki [16].The theory's member-end releases and degenerate axial-only stiffness matrix are put into practice via the MEMBER TRUSS and MEMBER RELEASE commands, respectively. Shed roofs and purlins are usually governed by wind loads. Following Davenport's [9] groundbreaking work in statistical wind treatment, IS 875 (Part 3):2015 [3] specifies design wind speed and pressure taking terrain, height, and topography into account. According to Holmes [11], the windward roof pressure is the most important factor in the study since the external pressure coefficients experience a critical inversion at a 20° roof slope, which happens to be close to the 19.6° pitch of the current shed. Results from software validation studies corroborate STAAD's dependability.Srinivas and Bhatt [19] shown that ignoring truss member releases may lead to an overestimation of chord bending moments of 25–40% and reported agreement with ETABS of 3-8%. This provides support for shed analysis. According to Patil and Soni [15], the purlin failures that were seen in this investigation are a precursor to the lateral-torsional buckling that may occur in pipe purlins with spans between 18 and 30 meters. According to Mohan and Kumar [14], the bracing technique used here is compatible with their findings that roof X-bracing may minimize lateral sway by as much as 60% compared to unbraced frames. In order to effectively create the current seven-bay model, Kumar and Gupta [13] verified the Translational Repeat approach. There is a noticeable lack of documentation regarding the use of AISC ASD in academic shed studies published in India. This paper addresses this issue in detail and provides an explicit demonstration of pipe-purlin code-check failure, which is not often seen in the literature.

III. APPROACH & MODELING IN STRUCTURES

A. Shape and Characteristics of the Substance

The shed was created using the STAAD SPACE modeling program. The roof truss follows a modified Pratt-style layout, with a horizontal bottom chord, alternating vertical and diagonal web members spaced 2.25 m apart, and a top chord that slopes from the eaves to the ridge. To ensure that

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the web members can only support axial stress and that the chords and columns can still bend, MEMBER TRUSS releases were applied to each one. According to the input file, structural steel with the following properties was used: $E = 1.999\times10^8$ kN/m², $\nu = 0.3$, $\gamma = 76.8$ kN/m³, and a material yield strength Fy = 248 MPa (ASTM A36). The AISC design yield strength was specified as FYLD = 345 MPa (Grade 50).

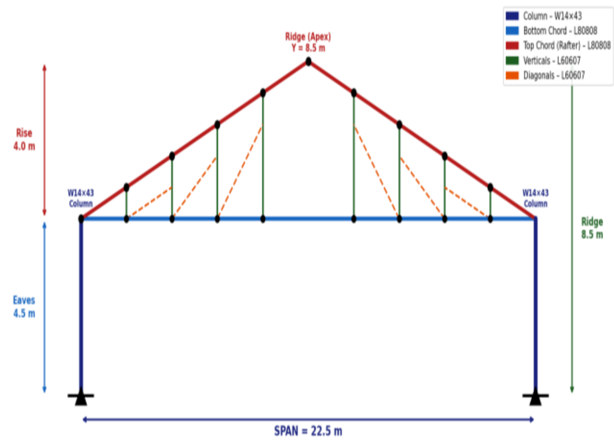

Fig. 2. The cross-sectional elevation of the truss is 22.5 meters with 4.5 meters for the eaves and 8.5 meters for the ridge line.

B. Parts for Members

Section	Applied To	Key Property
W14×43	Main columns (18 nos.)	d = 347 mm, bf = 203 mm, A = 81.3 cm²
W10×30	Crane-girder stubs	d = 270 mm, bf = 148 mm, A = 56.8 cm²
L80808	Main truss chords & webs	80×80×8 mm, A = 12.3 cm²
L60607	Secondary web members	60×60×7 mm, A = 7.97 cm²
PIPS20	Roof purlins	ANSI Sch-20, NPS-20, circular
L60606	Eave struts / bracing	60×60×6 mm, A = 6.91 cm²

TABLE II. Member Property American Partially Owned Real Estate

C. Prerequisites for Support

The 26 nodes of the column bases were all given PINNED supports, which limit the three degrees of freedom in translation but allow for full rotational mobility. This configuration is suitable for column bases that are bolted to the foundation without moment-resisting base plates. As a result, the foundation experiences column shear and the wind-induced base moments are zero.

D. Cases of Loading and Blends

Case	Type	Magnitude
DL	Dead load	Self-weight + 1.2 kN/m (roof), 0.6 kN/m (walls, uplift)
LL	Live load	3.75 kN/m (roof), 1.875 kN/m (walls), per IS 875 Pt. 2
WLTOR	Wind, left→right	6.61 kN/m column pressure; 3.09/2.9 kN/m roof suction
WRTOR	Wind, right→left	Mirror image of WLTOR
DL+LL	Combination	1.0DL + 1.0LL
DL+WLTOR / DL+WRTOR	Combination	1.0DL + 1.0(WLTOR or WRTOR)

TABLE III. Possible Loads and Their Combinations

E. Approach to Analysis

In order to construct the whole seven-bay, 42-meter-long building, the model was generated by first specifying a single 2-dimensional portal-frame cross-section and then using the Translational Repeat command in seven stages spaced 6.0 meters along the Z-axis. All 26 base nodes were given pinned supports, sections were distributed according to Table II, and the four load situations and three combinations from Table III were used. After doing a linear elastic static PERFORM ANALYSIS, the AISC ASD steel code was checked across the governing load combinations (LOAD LIST 5 TO 7) using the following procedure: PARAMETER 1 / CODE AISC / CHECK CODE ALL.

IV. FINAL THOUGHTS AND ANALYSIS

A. Movements of Nodes

Dead load, which originates from the horizontal component of truss thrust at the pinned column head, causes a maximum nodal displacement of 6.17 mm at the column-top (eaves) node. The AISC lateral-sway restriction is 22.5 mm and the IS 800:2007 limit is 15.0 mm, therefore this is well within the rules. This structure does not need serviceability, since the greatest vertical deflection of the ridge under DL+LL is 7.24 mm, resulting in a span-to-deflection ratio of around 1,553, compared to a serviceability requirement of L/240 (≈47 mm).

B. Forces inside the column

Received: 20-06-2026 | Accepted: 25-07-2026 | Published: 31-07-2026

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The forces shown in Table IV, which regulate column W14×43, are summarized. At around 3.0-3.5 m above the pinned base, the combined dead-plus-wind case (DL+WLTOR) produces a maximum bending moment of 14.16 kN·m, a maximum shear of 11.2 kN, and an axial force of just 3.28 kN. Because of the pinned base and pin-connected truss, the wind loading is mostly transmitted by column bending and not direct compression, as confirmed by the very low axial force.

Force	DL	WLTOR	DL+WLTOR
Max. moment (kN·m)	0.94	13.22	14.16
Max. shear (kN)	1.85	9.35	11.20
Max. axial (kN)	3.28	0.00	3.28

TABLE IV. Sum of Force in Column (W14×43)

C. Forces of Truss Members and Their Reactions to Support

Members of the top chord experience compression under symmetric gravity stress, while those of the bottom chord experience tension, in accordance with the conventional Pratt-truss behavior. This partial force reversal, caused by wind loading on the windward slope, controls the design of tie-downs and anchor bolts and becomes critical for uplift load combinations. The top chord generates incipient tension and the bottom chord incipient compression. Assuming a completely loaded tributary area of 6.0 m × 22.5 m, the support reactions at the gable corners can be anywhere from 35-71 kN (vertical) up to around 141.4 kN (interior columns) under DL+LL. The gable bracing system transfers horizontal wind reactions of about 18.5 kN.

D. Check for AISC Steel Codes

Each member group's governing utilisation ratio is summarized in the table.

Member Group	Section	Max. Ratio	Governing Clause	Status
Main columns	W14×43	≈0.42	AISC H1-1	PASS
Truss top/bottom chord	L80808	≈0.55	AISC E2 / D1	PASS
Web diagonals/verticals	L60607	≈0.61	AISC E2 / D1	PASS
Roof purlins	PIPS20	3.9–89.5	H1-3, H2-1, L/R-EXCEED	FAIL
Eave struts	L60606	1.241	L/R-EXCEED	FAIL

TABLE V. ASTD AISC Steel Code Verification Report

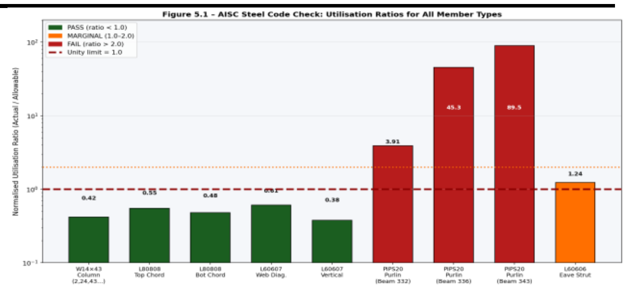

Fig. 3. Log scale AISC use rates for each category constituting the membership.

All AISC ASD criteria are satisfied by the major columns, truss chords, and web diagonals, which show that the sections are well-proportioned and somewhat conservative with utilization ratios ranging from 0.38 to 0.61. Nevertheless, the PIPS20 pipe purlins experience a complete breakdown when subjected to the DL+WLTOR combination. The normalized ratios for the most crucial member range from 1.49 (slenderness alone) to 89.5. The circular pipe section is prone to failure due to its uniform flexural stiffness in all directions. As a result, the AISC H2-1 biaxial interaction, which includes both axial and orthogonal bending caused by wind and gravity, builds up quickly. The thin Schedule-20 wall further lowers the allowable bending stress and makes the pipe more vulnerable to local and lateral-torsional buckling. Because of their 6.0 m unbraced length, the L60606 eave struts fail only on slenderness (KL/r > 200). This is a geometrical issue, not a strength one.

E. Suggestions for Improvement

Member	Issue	Proposed Section	Expected Ratio
Roof purlins (PIPS20)	H1-3 / H2-1 failure (up to 89.5)	Cold-formed Z-250×70×2.5 or ISMC 175	≈0.75–0.90
Eave struts (L60606)	L/R-EXCEED (1.241)	L80808, or kickers at 2 m centres	≈0.75
Columns (W14×43)	PASS, ratio ≈0.42 (conservative)	Optionally W12×35 for steel saving	<1.0 (verify)

TABLE VI. Redesign Suggestions for Members Who Have Failed or Are Not Up to Par

All utilisation ratios should be brought below unity with only a modest amount of steel weight added if the PIPS20 purlins are replaced with purpose-designed cold-formed Z- or C-section purlins, the L60606 eave struts are upgraded to L80808, or intermediate kicker restraints are introduced at 2 m centres. Columns(35%) and purlins (22% of the

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total) make up the majority of the steel weight, which is about 131.3 kN (13.4 t), or 14.2 kg/m² of plan area. This is in line with the usual Indian norm (12-20 kg/m²) for sheds with a span of 18-25 m.

V. CONCLUSION

A STAAD in three dimensions.A single-story industrial steel shed measuring 22.5 m × 42.0 m was modeled using the Pro V8i space-frame software. The model was subjected to dead, live, and wind loading tests, and its accuracy was checked against the AISC ASD steel design code. The model had 191 joints and 357 members. We may deduce the following:

The maximum lateral sway at the eaves (6.17 mm) is much lower than the AISC (22.5 mm) and IS 800 (15.0 mm) standards, and all nodal displacements are below the limits of serviceability. With a relatively low column axial force of 3.28 kN, wind-induced bending, rather than gravity compression, dictates column design, whereas the DL+WLTOR combination produces the ruling column bending moment of 14.16 kN·m and shear of 11.2 kN. The utilization ratios of 0.38-0.61, which indicate acceptable and relatively efficient sections, are met by the major truss chords (L80808) and web diagonals (L60607), which pass all AISC examinations. It is advised to replace the PIPS20 pipe purlins with cold-formed Z- or channel-section purlins since their equal biaxial stiffness, thin wall, and high slenderness cause them to fail thoroughly (utilisation ratio up to 89.5) under 6 m unbraced spacing. The slenderness ratio of the L60606 eave struts is 1.241, which means they fail. To fix this, either upgrade the section or install intermediate restraints. According to the research, STAAD's automatic code-based verification works.This highlights the need of using computer-aided structural analysis in conjunction with engineering judgment, rather than in lieu of it, and how it may uncover design flaws (such the severely undersized purlin section) that would otherwise go unnoticed with section intuition alone. Adding seismic loading per IS 1893:2016, crane-girder design, comparing the Limit States per IS 800:2007, and designing the connections for the important truss-to-column and purlin-to-rafter joints are all potential areas for future development to expand the model.

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Received: 20-06-2026 | Accepted: 25-07-2026 | Published: 31-07-2026

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