STRUCTURAL DESIGN OF A STEEL WAREHOUSE COMPOSED OF PORTAL FRAMES FORMED BY ROLLED STEEL PROFILES

João Victor Cordeiro Barros, Douglas Mateus de Lima, and Mateus Narcizo de Almeida Nunes
Technology Center, Academic Center of Agreste, Federal University of Pernambuco, Marielle Franco Avenue, Nova Caruaru, Caruaru-PE, 55014-900, Brazil
joao.cordeirobarros@ufpe.br
douglas.mlima@ufpe.br
mateus.narcizo@ufpe.br

ABSTRACT

In Brazil, the use of steel as a structural element in the most diverse segments of civil construction is increasingly frequent, especially in the industrial area, where there is a significant use of this material, particularly in the construction of warehouses. Nevertheless, when considering the manufacture of warehouses with portal frames structured in laminated steel profiles, it is noticeable that their adoption is relatively limited, while in other countries, where there is a greater consolidation of the steel culture, the use of this solution is widely spread, mainly due to its speed and practicality in the execution of projects. Therefore, the objective of the present work is to develop the structural design of a warehouse composed of laminated frames, to be implemented in the municipality of Belo Jardim, in the Agreste region of Pernambuco, in order to provide a wider range of solutions for this segment in the region. To this end, the structural elements were initially dimensioned, in unit form, through manual calculations, and subsequently their modeling and structural analysis were performed using the SCIA Engineer 22.0 software (student version), according to the current standards. Finally, the results of the structural design and the analysis of the quantity required for the construction project are presented.

Keywords: *Warehouse; Steel Structure; Laminated Steel Profiles; Structural Design.*

I. INTRODUCTION

Despite the predominance of reinforced concrete structures in Brazil, the use of steel structures in current constructions is becoming increasingly noticeable, ranging from small residential buildings to large-scale projects such as bridges and skyscrapers. In its use as a structural element, steel offers a range of characteristics that, compared to other conventional construction materials, constitute increasingly exploited advantages. Among them, the following can be mentioned: high strength, resulting in elements with smaller cross-sectional dimensions; high ductility, providing resistance to impacts; a high degree of reliability as it is an isotropic and homogeneous material; as well as ease of reinforcement and expansion, allowing greater flexibility in constructions [1].

Due to its high strength-to-weight ratio, steel enables the design of lighter structures, consequently allowing the construction of larger spans. As a result, it facilitates the execution of projects with a large covered area and ample interior space, such as warehouses, which are typically built using steel or precast reinforced concrete elements, often consisting of a single floor, used for various purposes in which these characteristics become essential, such as industrial, commercial, and sports facilities etc.

Warehouses are highly requested constructions in the Agreste region of Pernambuco [2]. In terms of steel typology, there is a greater use of trusses made of cold-formed steel profiles, which is directly related to their lower weight and, therefore, to the lower cost of obtaining steel, compared to the one made of rolled profiles [3]. However, in constructions where time is a crucial factor, such as distribution centers and hospitals, the type consisting of rolled profiles offers a considerable advantage. This is because it presents as the main characteristic the speed of installation, besides providing ease of fabrication and transportation, as well as a lower maintenance cost against the oxidation of painted elements, for example, due to its smaller contact surface with the environment and lesser amount of constructive details, especially links, susceptible to corrosion [4].

Thus, the justification for this study is precisely to provide the job market with the results of the structural design of a warehouse composed of frames formed by rolled profiles [5], dimensioned according to ABNT NBR 8800: 2008 [6] and ABNT NBR 14762: 2010 [7], aiming to promote its greater participation in current constructions and provide a wider range of possibilities of structural solutions for warehouses in the region. To this end, the following are presented: the methodology (project flowchart, building description, materials used, loads, load combinations, software used, structural layout); results and discussions of the main structural elements (purlins, portal frame rafter, columns, tie beams, vertical and roof bracing systems, bridging and connections, and quantity of materials); as well as the main conclusions of this research.

II. METHODOLOGY

In this section, the initial characterization of the project is presented, including information such as the description of the building, materials used, structural layout, determination of the applied loads, and combinations related to the limit states. Figure 1 illustrates a flowchart outlining the steps in the project development.

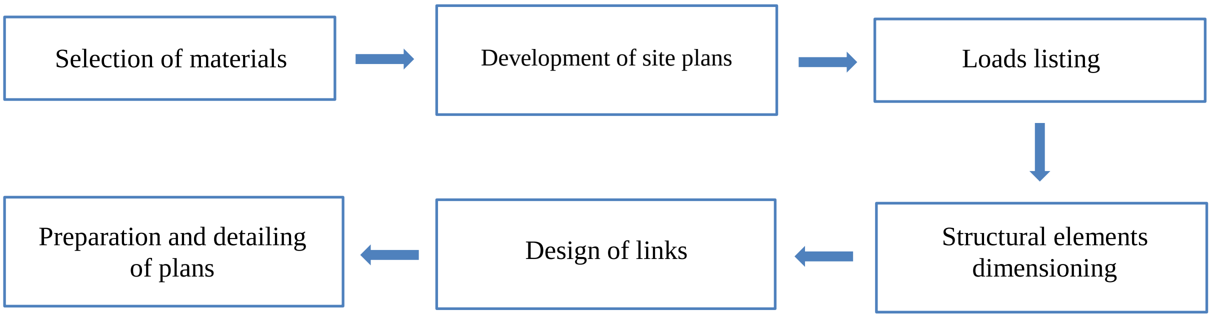


Figure 1. Project flowchart

2.1. Building Description

The building consists of an industrial warehouse designed to be built next to the BR-232 highway, in the municipality of Belo Jardim, in the Agreste of Pernambuco. As for its characteristics, it is a twinned warehouse with double-pitched roofs, with a length of 100 m, a width of 40 m, a free span of 20 m each, and a ceiling height of 6 m.

2.2. Materials Used

For the roofing and side enclosure of the warehouse, ISOTELHA® ALUMINIUM TRAPEZOIDAL PIR AP thermally insulated tiles with 30 mm thickness of insulation material are used [8]. The materials of the other elements are specified in Table 1.

Table 1. Materials and their specification.

Materials	Specification	Used for
Rolled profiles	ASTM A572 Gr. 50	Beams, Columns, and Bracing System
Cold-formed steel profiles	CF 26	Purlins and Bracing System
Connection plates	MR 250	Connections
Bolts and nuts	ASTM A325	Connections
Welding electrode	AWS E-7018	Connections
Concrete and grout	C-25	Foundation

2.3. Loads

In the structural analysis, it is necessary to define the loads that will act on the structure, in order to consider their influences in producing significant effects on the structural elements, based on the ultimate and serviceability limit states. Thus, the ABNT NBR 6120:2019 [9] and ABNT NBR 6123:1988 [10] standards, as well as [6] and [7], were considered in the determination of the loads.

2.4. Load Combinations

The load combinations for the ultimate and serviceability limit states are presented in Tables 2 and 3, respectively. Regarding the load factor coefficients, according to the ABNT NBR 8800:2008 [6] and ABNT NBR 14762:2010 [7] standards, combination and reduction factors must be considered for the calculation of the combinations related to the respective limit states, whose values can be found in these standards.

Table 2. Ultimate limit state load combinations

Load Combination	Type	Weighted loads
1	Normal	$1.25G_{per} + 1.50Q_{sc}$
2	Normal	$1.25 \cdot G_{per} + 1.40 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
3	Normal	$1.25 \cdot G_{per} + 1.40 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)
4	Normal	$1.25 \cdot G_{per} + 1.50 \cdot Q_{sc} + 1.40 \cdot 0.60 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
5	Normal	$1.25 \cdot G_{per} + 1.50 \cdot Q_{sc} + 1{,}40 \cdot 0.60 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)
6	Normal	$1.25 \cdot G_{per} + 1.50 \cdot 0.70 \cdot Q_{sc} + 1.40 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
7	Normal	$1.25 \cdot G_{per} + 1.50 \cdot 0.70 \cdot Q_{sc} + 1.40 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)
8	Normal	$1.0 \cdot G_{per} + 1.40 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
9	Normal	$1.0 \cdot G_{per} + 1.40 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)

Table 3. Serviceability limit state load combinations

Load Combination	Type	Weighted loads
10	Quasi-permanent	$1.0 \cdot G_{per} + 0.4 \cdot Q_{sc}$
11	Rare	$1.0 \cdot G_{per} + 1.0 \cdot Q_{sc} + 0.3 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
12	Rare	$1.0 \cdot G_{per} + 1.0 \cdot Q_{sc} + 0.3 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)
13	Rare	$1.0 \cdot G_{per} + 0.6 \cdot Q_{sc} + 1.0 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
14	Rare	$1.0 \cdot G_{per} + 0.6 \cdot Q_{sc} + 1.0 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)
15	Rare	$1.0 \cdot V_{0°}$ (c_{pi} = +0.2 or -0.3)
16	Rare	$1.0 \cdot V_{90°}$ (c_{pi} = +0.2 or -0.3)

2.5. Software Utilized

For the preliminary dimensioning of the structure, MathCad Prime 7.0 [11] software was initially used for the mathematical development of the loads acting on the elements and the verifications. The Ftool [12] software was used to perform structural analysis of the main elements of the warehouse in an isolated manner, as it provides greater ease in inputting loads and visualizing internal efforts and corresponding deflections.

For the modeling and analysis of the entire structure, the SCIA Engineer 22.0 [13] software was used, which consists of a finite element program developed to provide integrated modeling, analysis, and design for all types of structures.

2.6. Structural Layout

Regarding the structural layout of a warehouse with a low-slope roof, the proper positioning of the structural elements should consider [14]:

I – The spacing between the purlins is determined by the allowable span of the roof panels, as recommended by the manufacturer. In this case, the maximum span between the roof tile supports is 2200 mm, so the distance between the purlins is set between 1772 mm and 1950 mm;

II – For purlins made of cold-formed profiles, a maximum span of 7 m is recommended. In this case, the span is 5 m, corresponding to the distance between the main portal frames.

III – Considering the roof slope, the purlins work in oblique bending, in addition to axial stress due to the load of the wind in the building, resulting in composite and oblique bending. Thus, due to their reduced stiffness in the direction of the minor axis, bridging was positioned at each third of the purlins, in order to reduce their unbraced length in this direction;

IV – For the vertical closure panels, closure strips are required for fixing the tiles, with the spacing according to the manufacturer's recommendations. For cases of small spans, cold-formed profiles are usually used, otherwise, laminated I-type profiles are used;

V – The bracing system of the roof is arranged in an X-shape, which should be designed to ensure the bracing of the compressed elements, as well as the transmission of horizontal stresses to the structure's supports. It is recommended that the distance between the bracing planes does not exceed 20 m, and they should be provided at the edges of the structure to transmit wind loads.

VI – The vertical bracing is arranged in an X-shape and is used to ensure the overall stability of the structure, usually positioned at every three main portal frames.

Thus, considering the fulfillment of the mentioned criteria, the structural layout was carried out as shown in Figure 2.

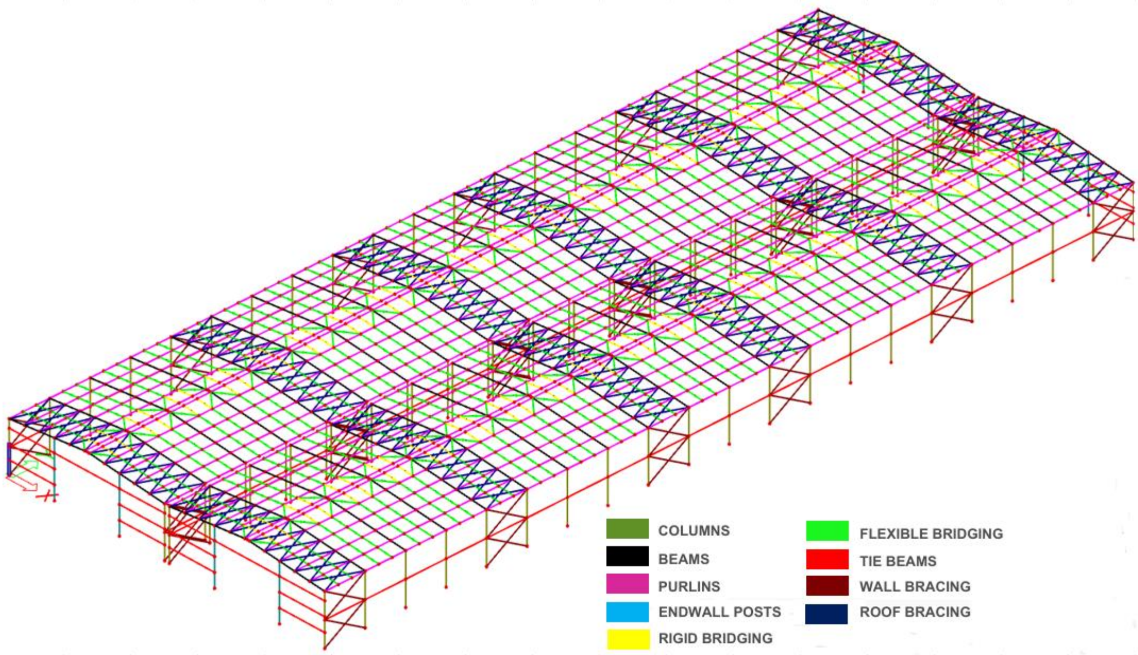


Figure 2. 3D view of the warehouse

III. RESULTS AND DISCUSSIONS

In this section, the internal stresses and checks of the structural elements of the warehouse are presented. In addition, the connections between the elements that make up the structure and the connections with the foundation are presented, as well as the survey of the quantity of materials.

3.1. Purlins

The simply supported structural model was used, adopting the stiffened U-type profile (Ue 100x50x17x3.0), as recommended in ABNT NBR 6355:2012 [15]. The verifications followed the recommendations of ABNT NBR 14762:2010 [7], evaluating the mechanisms of local buckling, onset

of yielding and global buckling by bending, torsion, flexural and distortional buckling, using the effective section method (ESM) and the effective width method (EWM), for the x and y axes, respectively.
The combined stress analysis considered the simultaneous action of bending and shear force, as well as bending and normal stress. Table 4 shows the internal forces calculated for the most stressed purlin, obtaining a 90% utilization.

Table 4. Internal forces on purlins

Element	N_d (kN)	M_{dx} (kN.m)	M_{dy} (kN.m)	V_{dx} (kN.m)	V_{dy} (kN.m)	Utilization (%)
Purlin	-1.13	3.79	0.00	0.05	2.87	90.00

The serviceability limit state checks shall follow the recommendations of Annex A of ABNT NBR 14762:2010 [7], concerning the maximum vertical displacements. For roof purlins, the vertical displacement is limited to L/180 for rare service combinations with variable loads in the same direction as the permanent load, and L/120 for variable loads in the opposite direction of the permanent load, where L is the length of the purlin.
Thus, for 5 m purlins, the limiting values are 27.77 mm and 41.67 mm, the largest displacements, obtained in SCIA Engineer 22.0, being 17.2 mm and 20.5 mm, respectively.

3.2. Portal Frame Rafter

Due to its slope, which follows the plane of the roof, the frame beams are elements that work under normal composite bending, as shown in Figures 3A, B, and C, in which the diagrams of the most demanding internal forces of the same typical frame obtained through the analysis in Ftool and SCIA Engineer 22.0 are presented. It is noted that, despite the difference between the values obtained for the internal forces in both software, mainly due to the simplifications adopted in the analysis in Ftool, particularly regarding load transfer between elements, the results were satisfactory, with very close results also reached in the analysis of the other elements of the warehouse.
For the frame rafters, I-type profiles (W360x32.9) [16, 17] were used, which, in addition to providing a high stiffness in the plane of the greatest stresses, that is, in the plane of the frame, has a low linear weight, obtaining a maximum use of the element with the lowest possible weight.
The ultimate limit state checks should follow the recommendations of ANBT NBR 8800:2008 [6]. The mechanisms of web local buckling (WLB) and compressed flange were evaluated, as well as lateral torsional buckling (LTB), obtaining resistant design bending moments of 137.74 kN.m and 21.57 kN.m for the x and y axes of the profile, respectively, as well as global buckling by bending, resulting in resistant normal stress of 811 kN. The analysis of the combined forces was evaluated with the simultaneous stress of the normal force and bending. The serviceability limit state checks shall follow the recommendations of Annex C of the standard, which, for roof beams, limits the vertical displacement to L/250, L being its span. Since the free span is 20 m, the maximum vertical displacement is 80 mm. It should be noted that this was the lightest profile to meet the vertical displacement requirements.
The internal design forces and vertical displacements of the frame beams are presented in Table 5. Given the symmetry of the warehouses, results for only one of them will be presented, as the results are similar. It is noteworthy that, due to the low values of torsional forces in some elements, they were not considered in the beam verifications, and this consideration was also applied in the analysis of the other structural elements, such as the tie beams and the columns.

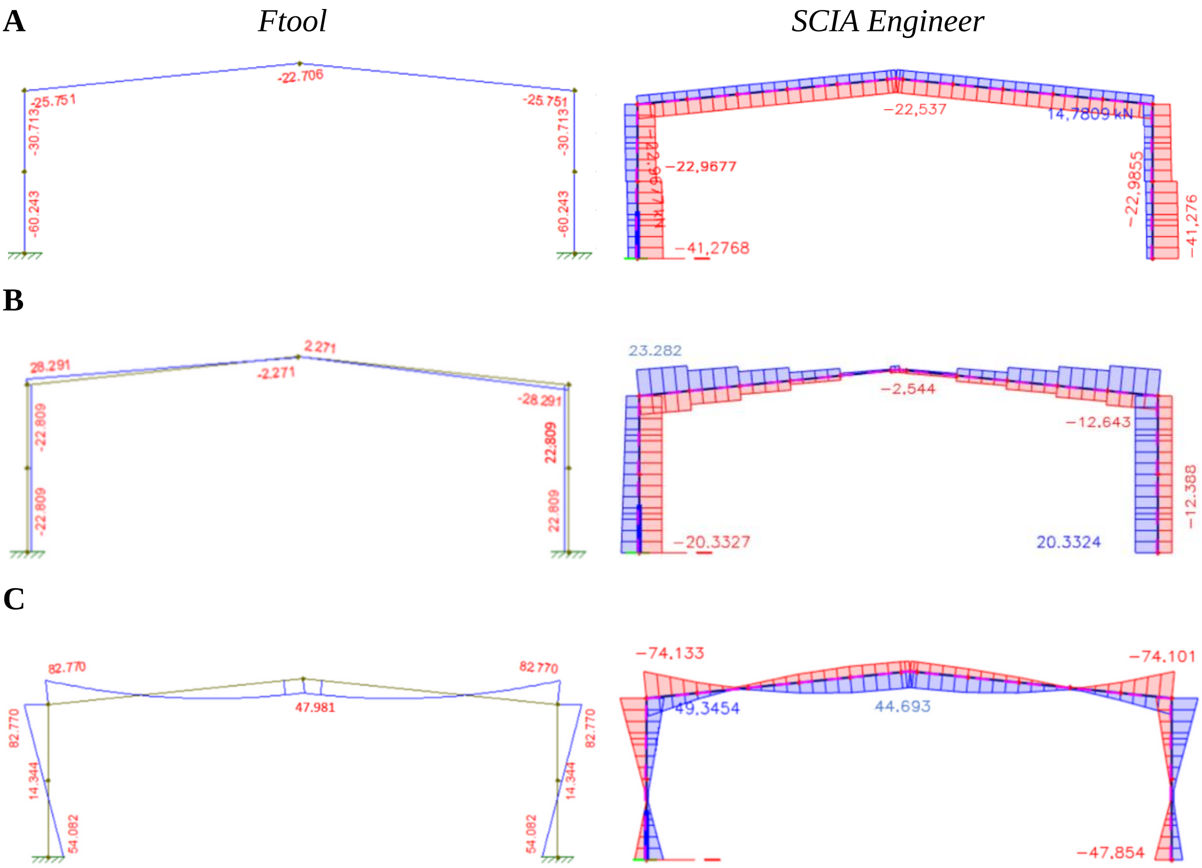


Figure 3. A. Normal stress (kN). B. Shear stress (kN). C. Bending moment (kN.m).

Table 5. Verification of portal frame rafters

Element	N_d (kN)	M_{dx} (kN.m)	M_{dy} (kN.m)	V_{dx} (kN)	V_{dy} (kN)	T_d (kN.m)	Utilization (ULS) (%)	δ (mm)	Utilization (SLS) (%)
V_{A1}	16.80	10.23	0.6800	0.38	3.50	0.0098	11.00	2.90	3.62
V_{A2}	-22.80	62.73	0.1700	0.11	21.40	0.0045	48.00	59.70	74.62
V_{A3}	-23.00	74.20	0.0000	0.00	23.30	0.0026	45.00	73.40	91.75
V_{A4}	-23.00	74.04	0.0016	0.01	23.30	0.0000	45.00	73.20	91.50
V_{A5}	-23.00	73.18	0.0392	0.02	23.30	0.0000	55.00	72.90	91.12
V_{A6}	-23.00	74.56	0.0241	0.01	23.60	0.0000	56.00	74.70	93.37
V_{A7}	-23.00	74.07	0.0043	0.01	23.30	0.0000	45.00	73.20	91.50
V_{A8}	-22.90	73.76	0.0049	0.01	23.20	0.0000	44.00	72.90	91.12
V_{A9}	-23.10	73.70	0.0016	0.01	23.40	0.0000	55.00	73.60	92.00
V_{A10}	-23.20	74.01	0.0090	0.01	23.50	0.0000	55.00	74.00	92.50
V_{A11}	-23.00	74.06	0.0067	0.01	23.30	0.0000	45.00	73.20	91.50
V_{A12}	-23.10	74.01	0.0071	0.01	23.50	0.0000	55.00	73.90	92.37
V_{A13}	-23.10	73.70	0.0014	0.01	23.40	0.0000	55.00	73.60	92.00
V_{A14}	-22.90	73.76	0.0014	0.01	23.20	0.0000	44.00	72.90	91.12
V_{A15}	-23.00	74.04	0.0055	0.01	23.30	0.0000	45.00	73.20	91.50
V_{A16}	-23.30	74.56	0.0207	0.01	23.60	0.0000	56.00	74.60	93.25
V_{A17}	-23.00	73.19	0.0364	0.01	23.30	0.0000	55.00	72.90	91.12
V_{A18}	-23.00	74.04	0.0083	0.00	23.30	0.0000	45.00	73.20	91.50

V_{A19}	-23.00	74.20	0.0099	0.00	23.30	0.0028	45.00	73.40	91.75
V_{A20}	-22.80	62.73	0.168	0.11	21.40	0.0043	48.00	59.70	74.62
V_{A21}	16.70	10.23	0.6800	0.38	3.50	0.0098	11.00	2.90	3.62

3.3. Columns

Responsible for transferring the loads to the foundation, the columns are elements requested both in the plane of the frame and out of it, due to the frontal wind loading in the building, thus, they are subjected to composite and oblique bending. Therefore, the H-type profile was used, being W250x62 (HP) for the main portal frames, located on the sides of the warehouses, and W200x35.9 (H) for the endwall posts, located on the front and rear faces, both being fixed at the base and with rigid and flexible connections at the top, respectively.

Table 6 shows the internal design forces. The verifications of the columns also followed the recommendations of ABNT NBR 8800:2008 [6], and the same mechanisms mentioned in 3.2 were evaluated. Regarding the columns of the portal frames, the checks resulted in a resistant normal force of 1849 kN, and resistant bending moments of 221.53 kN.m and 94.77 kN.m to the x and y axes of the profile, respectively.

Table 6. Verification of the columns in the portal frames

Element	**N_d (kN)**	**M_{dx} (kN.m)**	**M_{dy} (kN.m)**	**V_{dx} (kN)**	**V_{dy} (kN)**	**T_d (kN.m)**	**Utilization (%)**
COLUMNS IN PORTAL FRAMES							
P_1	-31.50	8.81	1.85	1.48	4.08	0.0027	7.00
P_2	-27.40	69.40	0.0363	0.06	18.80	0.0154	32.00
P_3	-28.20	76.22	0.0386	0.06	20.60	0.0055	35.00
P_4	-28.00	74.97	0.0219	0.01	23.30	0.0034	35.00
P_5	-29.90	75.35	0.0167	0.14	20.40	0.0013	35.00
P_6	-28.70	75.86	0.0027	0.15	20.60	0.0000	35.00
P_7	-28.00	74.92	0.0017	0.15	20.30	0.0000	35.00
P_8	-28.00	74.84	0.0067	0.16	20.30	0.0000	35.00
P_9	-29.00	75.62	0.0057	0.16	20.50	0.0014	35.00
P_{10}	-28.60	75.62	0.0131	0.16	20.50	0.0019	35.00
P_{11}	-28.00	75.00	0.0125	0.16	20.30	0.002	35.00
P_{12}	-28.70	75.62	0.0124	0.16	20.50	0.0022	35.00
P_{13}	-28.60	75.63	0.0177	0.16	20.50	0.0027	35.00
P_{14}	-28.00	74.84	0.0188	0.16	20.30	0.0031	35.00
P_{15}	-28.00	74.92	0.0242	0.17	20.30	0.0036	35.00
P_{16}	-28.70	75.86	0.0263	0.17	20.60	0.0044	35.00
P_{17}	-29.10	75.35	0.0412	0.18	20.40	0.0058	35.00
P_{18}	-28.00	74.97	0.051	0.22	20.30	0.0079	35.00
P_{19}	-28.20	76.22	0.0686	0.26	20.60	0.0101	35.00
P_{20}	-27.40	69.41	0.0069	0.26	18.80	0.0199	32.00
P_{21}	16.60	0.665	7.82	3.90	2.53	0.0020	9.00
ENDWALL POSTS							
P_{22}	-8.71	20.18	0.0428	0.07	12.70	0.0000	18.00
P_{24}	-11.60	20.67	0.062	0.02	13.00	0.0000	18.00
P_{68}	-15.90	14.26	0.717	0.25	8.91	0.0000	15.00
P_{70}	-13.40	14.15	0.769	0.30	8.85	0.0000	14.00
P_{23}	-8.72	17.77	0.0419	0.07	13.40	0.0000	16.00

P_{25}	-11.70	18.26	0.061	0.01	13.70	0.0000	16.00
P_{69}	-11.70	18.34	0.0632	0.02	13.70	0.0000	16.00
P_{71}	-8.60	17.85	0.0437	0.07	13.50	0.0000	16.00

Regarding the serviceability limit state of the columns, ANNEX C of ABNT NBR 8800:2008 [6] recommends that the lateral displacement of the columns compared to the base is limited to H/300, where H is the height of the column. In this case, the frame columns have a height of 6 m, resulting in a maximum horizontal displacement of 20 mm. Figure 4 shows the deflection of a typical portal frame, with a displacement of 9.4 mm for the columns, achieving 47% of utilization. In Figure 4, it is noteworthy that the deflections of the beams are similar, with one beam omitted in order to improve visualization.

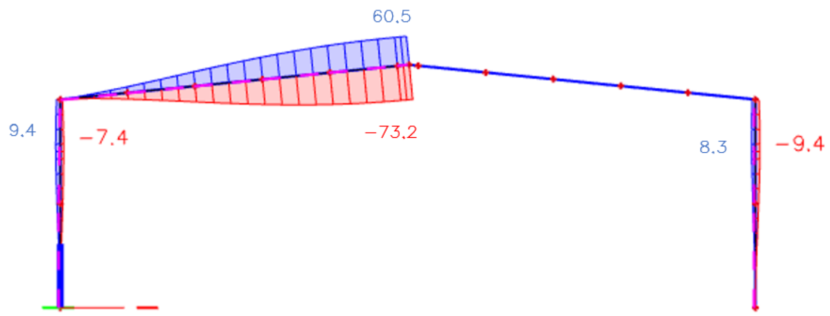


Figure 4. Deflection of a typical portal frame (mm)

In the case of the portal frame, although its utilization in relation to the resistant capacity is only 35%, this was the lightest profile available on the market to meet the displacement requirements of the frame, as shown in Figure 4. Regarding the endwall posts, despite the gap in their utilization, the choice was conditioned mainly to the connections, prioritizing having dimensions of the cross-section compatible with those of the tie beams.

3.4. Tie beams

The tie beams are subjected to oblique and composite bending due to wall and wind loads, as well as axial loads due to the frontal wind in the warehouse. H-type profiles were used, with W200x46.1 for the beam over the access gate, and W200x35.9 for the other tie beams.

Table 7 shows the internal forces, obtained based on ANBT NBR 8800:2008 [6], in which the same mechanisms presented in 3.2 were evaluated. It is noteworthy that, despite the gap, these were the lightest profiles and with greater availability in the market to meet the requirement of slenderness to compression in relation to the axis of lower inertia.

Also, in Table 7 are presented the results of the serviceability limit state analysis, verified based on ANNEX C of ANBT NBR 8800:2008 [6]. For beams supporting masonry walls, as is the case of the side closure, the vertical displacement should not be greater than 15 mm. In other cases, the displacement is limited to L/250. Being L equal to 5.82 m and 8.36 m for the front and rear closures and the beam over the gate, respectively, their displacements are limited to 23.28 mm and 33.44 mm, respectively.

Table 7. Verification of the tie beams

Element	N_d (kN)	M_{dx} (kN.m)	M_{dy} (kN.m)	V_{dx} (kN)	V_{dy} (kN)	T_d (kN.m)	Utilization (ULS) (%)	δ (mm)	Utilization (SLS) (%)
SIDE CLOSURE									
$V_{A(1-2)}$	-3.71	14.42	0.267	0.06	18.10	0.0027	15.00	4.20	28.00
$V_{A(2-3)}$	-6.91	14.35	0.0461	0.06	18.00	0.0000	15.00	4.20	28.00
$V_{A(3-4)}$	-5.25	14.30	0.00	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(4-5)}$	-3.60	14.29	0.0181	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(5-6)}$	-2.44	14.27	0.0215	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(6-7)}$	-3.23	14.22	0.0017	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(7-8)}$	-1.63	14.22	0.0122	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(8-9)}$	1.07	14.20	0.0082	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(9-10)}$	0.55	14.23	0.0861	0.02	18.00	0.0000	14.00	4.20	28.00
$V_{A(10-11)}$	-1.64	14.18	0.0111	0.01	18.00	0.0000	14.00	4.20	28.00
$V_{A(11-12)}$	-0.11	14.20	0.0762	0.01	18.00	0.0000	14.00	4.20	28.00
$V_{A(12-13)}$	-0.72	14.22	0.0239	0.02	18.00	0.0000	14.00	4.20	28.00
$V_{A(13-14)}$	-2.95	14.14	0.013	0.01	18.00	0.0000	14.00	4.20	28.00
$V_{A(14-15)}$	-1.48	14.14	0.0032	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(15-16)}$	-0.03	14.18	0.0183	0.00	18.00	0.0000	14.00	4.20	28.00
$V_{A(16-17)}$	-1.17	14.19	0.0154	0.01	18.30	0.0000	14.00	4.20	28.00
$V_{A(17-18)}$	-1.24	14.14	0.0236	0.01	18.00	0.0000	14.00	4.20	28.00
$V_{A(18-19)}$	-2.24	14.07	0.0432	0.01	18.00	0.0000	14.00	4.20	28.00
$V_{A(19-20)}$	1.86	14.14	0.335	0.07	18.00	0.0000	15.00	4.20	28.00
$V_{A(20-21)}$	2.04	14.34	0.0057	0.02	18.00	0.0012	14.00	4.20	28.00
FRONT AND REAR CLOSURE									
$V_{(A-B)1.1}$	0.00	2.44	5.33	0.35	0.22	0.0042	16.00	1.30	5.58
$V_{(A-B)1.2}$	-0.27	2.22	3.39	0.29	0.22	0.0017	11.00	1.20	5.15
$V_{(A-B)1.3}$	-3.74	23.10	41.80	0.35	0.22	0.0029	14.00	1.30	5.58
$V_{(B-C)1}$	-4.82	7.10	19.22	1.18	0.37	0.0000	38.00	5.80	17.34
$V_{(A-B)21.1}$	-0.15	1.71	0.00	0.00	0.37	0.0000	3.00	1.00	4.29
$V_{(A-B)21.2}$	0.46	1.71	0.00	0.00	0.37	0.0000	3.00	1.00	4.29
$V_{(A-B)21.3}$	-3.83	1.26	0.00	0.00	0.73	0.0031	3.00	1.10	4.72
$V_{(B-C)21}$	-4.92	4.48	0.00	0.00	0.71	0.0000	7.00	4.10	12.26

3.5. Vertical and Roof Bracing Systems

The bracing elements act both in the roof planes and in the lateral vertical planes, being spaced every 15 m at the ends, and 10 m in the center.

BR ½" profiles were used for the roof bracing, and L 100x4.25 equal flange angles for the lateral bracing, verified according to item 5.2 of ANBT NBR 8800:2008 [6] and item 9.6 of ABNT NBR 14762:2010 [7], respectively, and the axial traction stress mechanism was evaluated.

The biggest axial internal effort for the vertical and roof bracing was 12.70 kN and 19.50 kN, in this order, obtained through SCIA Engineer using a non-linear analysis, important for them to work only in traction.

3.6. Bridging

As mentioned in item 2.6, in order to reduce the unlocked length in the direction of lower inertia of the purlins, bridging was positioned at each third of their span. Profiles of the BR 1/2" type were used, acting only on traction, being called flexible bridging, as well as angle brackets with equal flaps L 50.8x3.18 in order to contain the last purlin of the ridge, called rigid bridging, working on compression. The maximum axial stress for the rigid and flexible bridging system was -0.05 kN and 0.37 kN, respectively, also obtained through a non-linear analysis in SCIA Engineer. It is noteworthy that the rigid bridging design had as a determining criterion the compression slenderness ratio, this being the lightest profile to meet this criterion.

3.7. Connections

This section presents the connections of the structural elements, calculated according to the recommendations of item 6 of ANBT NBR 8800:2008 for rolled profiles and item 10 of ABNT NBR 14762:2010, concerning cold-formed steel profiles.

3.7.1. Column-Foundation Connection

The base connections of the columns were considered as fixed connections, made using anchor bolts sized according to the AISC method [18], with the limit states as the analysis criteria, as well as the Gerdau® practical guide on connections for steel structures with rolled profiles [19].
Two types of fixed-end bases were designed, as shown in Figures 5A and B: one for the frame columns and one for the endwall posts, both subjected to bending moment and compression. In the design process, the mechanisms of plate crushing, traction, and shear in the anchor bolts were evaluated, as well as the concrete compressive strength and the anchorage length of the anchor bolts.

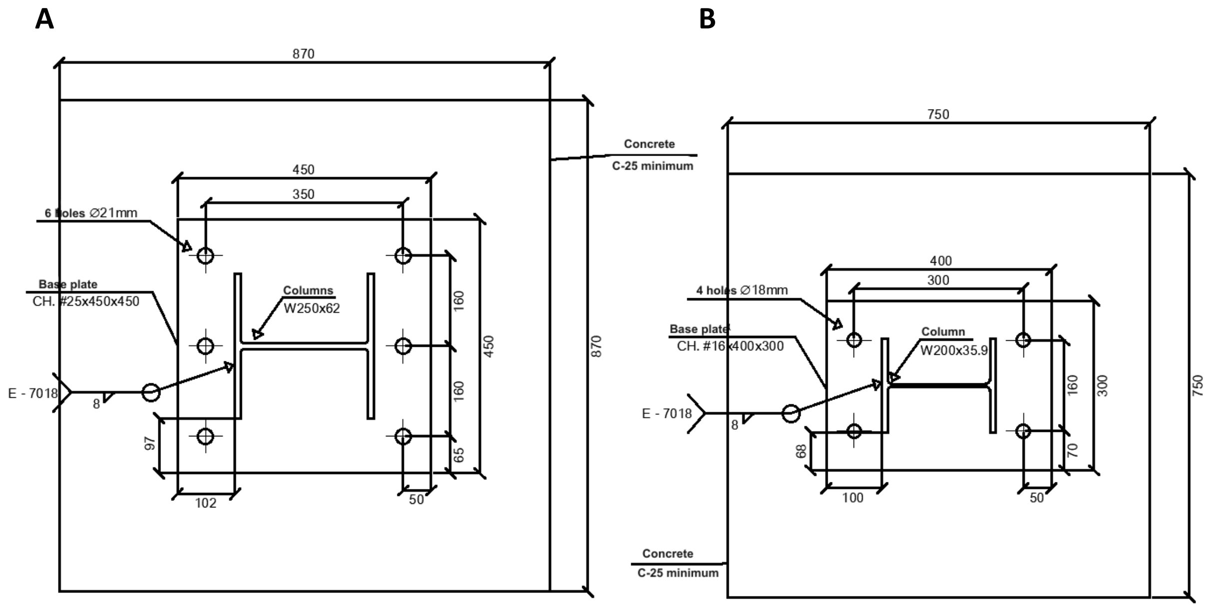


Figure 5. Connections of the columns. A. Main columns. B. Endwall posts (dimensions in mm)

3.7.2. Beam-to-Column Connection

The connections beam-to-column of the frames were considered rigid, to allow the "integral" transmission of the efforts between these elements. They were made by means of a header end plate, welded to the beam, and bolted to the flange of the column, as shown in Figure 6. The dimensioning took into account the criteria of rupture of the weld in the flanges and in the web, traction in the bolts, crushing, and flexural strength of the top cover plate. In addition to that, the verification of the tensile and compressed sections in the column table was performed, using 8.5 mm thick stiffeners. About the

stiffeners, it should be noted that in the definition of their dimensions, the spaces necessary for the fitting in the column to be subsequently welded should be discounted.
On the other hand, the connections between the tie beam and the column were considered flexible, working essentially with shear force. They were made by means of L 88.9x6.35 angles, with the web welded to the column and screwed to the beam web, as shown in Figure 7. Its design had as criteria the contact pressure, the collapse by tearing and the shear, both in the cantilever and in the beam web, as well as the rupture of the weld and the shear of the bolts.

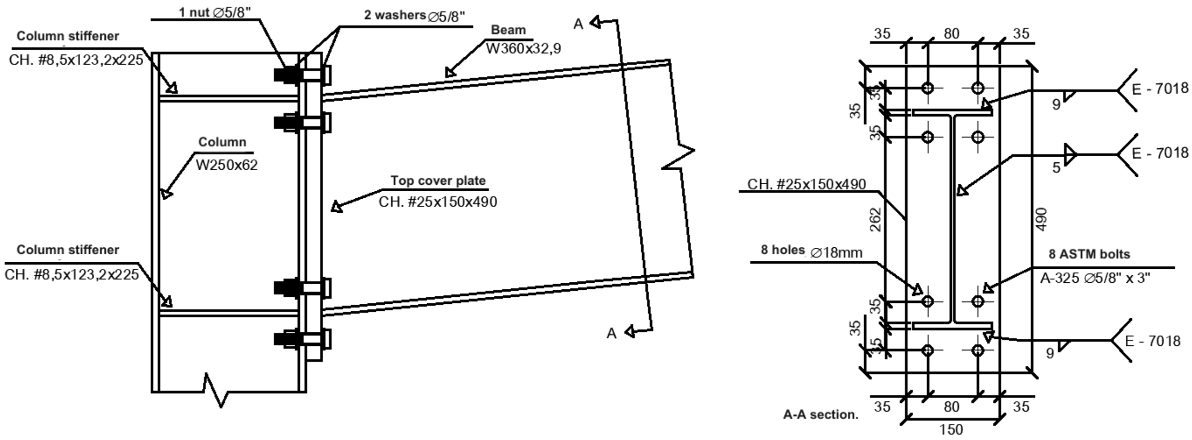


Figure 6. Beam-to-column connection in the main portal frames (dimensions in mm)

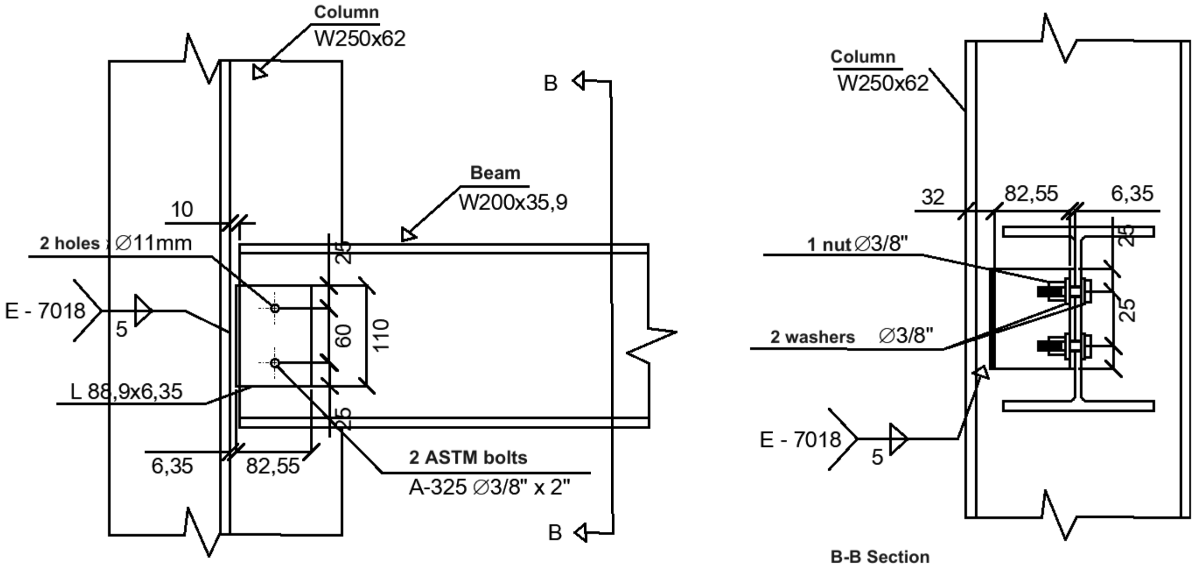


Figure 7. Beam-to-column connection in the tie beam (dimensions in mm)

3.7.3. Beam-to-beam Connection (roof ridge)

As well as the beam-column connections of the frames, the connections between beams in the ridge were considered rigid, made through a header end plate, welded to the beam, and bolted together. As observed in Figure 3, in the ridge, the efforts are predominantly axial, so that this form of connection allows the bolts to work essentially to traction, taking full advantage of one of the main characteristics of steel, which is its resistance to this effort.
Figure 8 shows the schematic of the ridge connection. In order to reduce the weight of the structure relative to the top cover plates, connection plates with centralized stiffeners on the beam flange (CH. #9.35x70x100) were considered, obtaining a reduction of 34.56% of the connection weight.

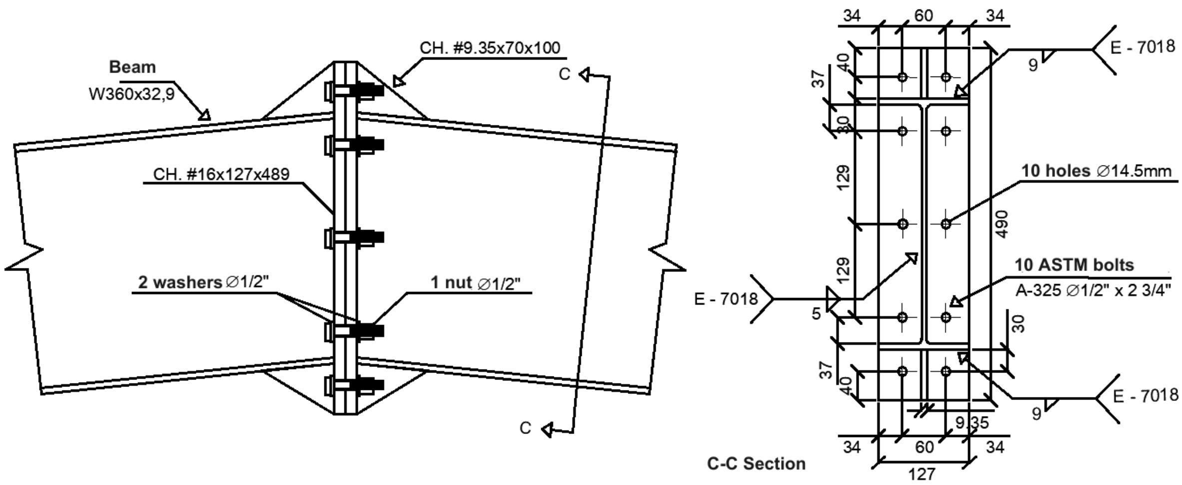


Figure 8. Beam-to-beam connection in the roof ridge (dimensions in mm)

3.7.4. Purlin to Beam Connection

The connections between the purlins and the beams were considered rigid, made using L 88.9x7.94 angle brackets welded to the beam and bolted to the purlin, also containing a central stiffener (CH. #4.76x83x83), as shown in Figure 9.
Due to the transmission of the bending moment from the purlins to the support, the bolts of the connection work essentially under shear stress. Thus, it was necessary to consider the reinforcement plate (CH. #7.94x50.8x80) in order to avoid the crushing of the purlin near the bolt hole.

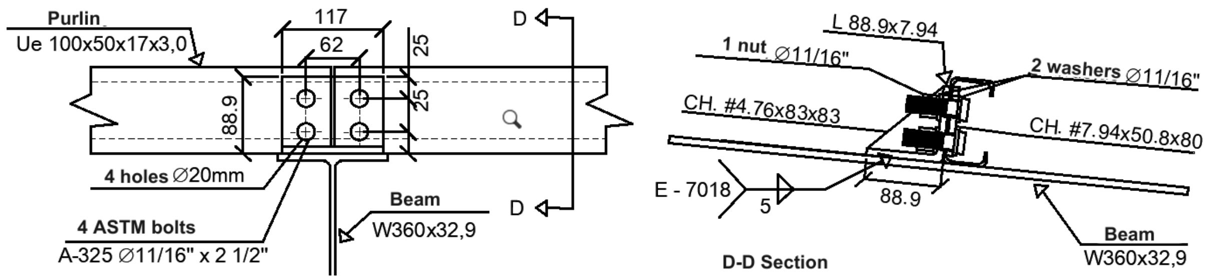


Figure 9. Purlin to beam connection (dimensions in mm)

3.8. QUANTITY OF MATERIALS

Tables 8 and 9 show the quantities of steel and the quantities of bolts, nuts, washers, and anchor bolts, respectively.

Table 8. Quantity of steel

ASTM STEEL A572 Gr. 50			
profile	unit mass (kg/m)	total length (m)	total mass (kg)
W360x32.9	32.9	847.56	27884.72
W250x62	62	504.00	31248.00
W200x35.9	35.9	592.92	21285.83
W200x46.1	46.1	33.43	1541.22
L 50.8x3.18	2.46	312	767.52
L 88.0x6.35	8.56	41.36	354.04
L 88.0x7.94	10.59	58.97	624.47
BR 1/2”	0.99	2946.80	2917.33
		total mass (kg)	86623.13
CF-26 STEEL			

profile	unit mass (kg/m)	total length (m)	total mass (kg)
Ue 100x50x17x3.0	5.05	2400	12120.00
L 100x4.25	6.44	559.78	3604.96
		TOTAL MASS (kg)	15724.96
MR 250 STEEL			
steel plate	unit mass (kg/m)	quantity (un)	total mass (kg)
#25x150x489	14.39	84	1209.17
#8.5x123.3x225	1.85	336	621.98
#16x127x490	7.80	84	655.21
#9.35x70x100	0.51	84	43.16
#4.76x83x83	0.26	504	129.74
#25x450x450	39.74	84	3338.21
#16x300x400	15.10	8	120.58
#4.76x181x153	1.03	192	198.68
#2.50x51x51	0.05	320	16.33
		total mass (kg)	6333.06

Table 8. Quantity of bolts, nuts, washers, and anchor bolts

BOLTS (ASTM A325)		
diameter (inch)	length (inch)	quantity (un)
1/2"	1 ¾”	320
1/2"	2”	384
1/2"	2 ¾”	420
3/8”	2”	432
5/8”	3”	672
11/16"	2 ½”	1920

NUTS (ASTM A325)	
diameter (inch)	quantity (un)
1/2”	1124
3/8”	432
5/8”	768
3/4"	1512
11/16”	1920

WASHERS			
flat		split lock	
diameter (inch)	quantity (un)	diameter (inch)	quantity (un)
1/2"	2248	-	-
3/8”	864	-	-
5/8”	1440	5/8”	64
3/4"	1512	3/4”	1008
11/16”	3840	-	-

ANCHOR BOLTS (ASTM A325)		
diameter (inch)	length (mm)	total length (mm)
5/8”	342	10944
3/4"	374	188496

IV. CONCLUSION

The structural design proved to be complex and required extreme caution in the choice of structural elements. This is because. as important as meeting the recommendation in the mentioned standards and manual regarding steel structures, economic considerations are also taken into account by minimizing the steel consumption in the structure.

Through the comparison of verifications in the main elements of the warehouse conducted using Mathcad Prime and Ftool, with those obtained from SCIA Engineer, it was observed that despite the simplifications adopted, the results were very close, as seen in Figure 9 for the case of a typical portal frame. This convergence of results is extremely important for validating the approaches used in the structural design.

However, one of the limitations in the development of the project was the analysis of the vertical bracing in SCIA Engineer 22.0. This is because, despite considering a non-linear analysis, in order to act only on traction, the software presented an error in global stability of the structure, being calculated as a standard element and having as results of its analysis axial compression efforts and bending moments, resulting in the choice of a more robust profile. Thus, future works involving warehouses developed in SCIA Engineer 22.0, as well as in other versions, may verify the presence of such inconsistency, and, if observed, analyze solutions in order to optimize the design.

A suggestion for future research is the development of structural projects of the same warehouse, with frames with full web steel profiles, with other software, to establish a comparative analysis with the results obtained in this work. As well as the design of the same warehouse with the truss typology (consisting of cold-formed profiles) aiming at the development of a comparative cost analysis and, thus, an economic feasibility analysis between the truss typologies and with full web steel profiles.

ACKNOWLEDGEMENTS

The authors would like to thank the Fundação de Amparo à Ciência e Tecnologia de Pernambuco (FACEPE), the Pro-Rectory of Graduate Studies (PROPG), and the Coordenação de Aperfeiçoamento de Pessoal de Nível Superior - Brazil (CAPES) - Finance Code 001, for funding the research developed in the Graduate Program in Civil and Environmental Engineering (PPGECAM) at the Federal University of Pernambuco (UFPE) at the Caruaru Campus.

REFERENCES

[1]. FAKURY, R. H.; SILVA, A. L. R. C.; CALDAS, R. B. Design of steel and composite steel and concrete structural elements. São Paulo: Person Education do Brasil, 2016.

[2]. SILVA, L. G. P.; LIMA, D. M. de; MEDEIROS, I. da S.; LÓPEZ-YÁNEZ, P. A. Structure steel warehouse solution. Research, Society and Development, [S. l.], v. 10, n. 8, p. e54810817765, 2021. ISSN 2525-3409.

[3]. SILVA, R. E. R. Steel structure warehouse design: steel consumption in solutions with rolled and cold-formed profiles. UFG - School of Civil and Environmental Engineering. Goiânia, 2017.

[4]. GERDAU. Collection of the use of steel: framed warehouses with rolled structural profiles. Available at: https://www2.gerdau.com.br/blog-do-aco/publication-category/manuais/? Accessed on: August 24, 2022.

[5]. DREHMER, G. A.; JÚNIOR, E. M.; PRAVIA, Z. M. C. Warehouses for general uses. 4. ed. Rio de Janeiro: Instituto Aço Brasil/Centro Brasileiro da Construção em Aço, 2010. 74 p. 1. Reprinted.

[6]. BRAZILIAN ASSOCIATION OF TECHNICAL STANDARDS – BRAZILIAN STANDARD. NBR 8800: Design of steel and composite structures for buildings. Rio de Janeiro: [s.n.], 2008. 237 p.

[7]. BRAZILIAN ASSOCIATION OF TECHNICAL STANDARDS – BRAZILIAN STANDARD. NBR 14762: Cold-formed steel design – Procedure. Rio de Janeiro: [s.n.], 2010. 87 p.

[8]. KINGSPAN ISOESTE. Catalog of products. 2022. Available at: https://kingspan-isoeste.com.br/catalogos/. Accessed on: August 24, 2022.

[9]. BRAZILIAN ASSOCIATION OF TECHNICAL STANDARDS – BRAZILIAN STANDARD. NBR 6120: Loads for structural design of buildings. Rio de Janeiro: [s.n.], 2019. 60 p.

[10]. BRAZILIAN ASSOCIATION OF TECHNICAL STANDARDS – BRAZILIAN STANDARD. NBR 6123: Building construction - Bases for design of structures - Wind loads - Procedure. Rio de Janeiro: [s.n.], 1998. 66 p.

[11]. PTC. Mathcad Prime 7. Desktop app (2023). https://www.mathcad.com/en/blogs/whats-in-prime-7

[12]. FTOOL. Ftool 2017. Desktop (2023). https://www.ftool.com.br/Ftool/

[13]. BIM Works. SCIA Engineer V22.0. Desktop app (2023). https://www.scia.net/en/scia-engineer/downloads?feature=scia-engineer-22

[14]. XEREZ NETO, J.; CUNHA, A. S. Steel structures: a practical manual for design, dimensioning and technical reports. 2. ed. São Paulo: Oficina de Textos, 2020. 448 p.

[15]. BRAZILIAN ASSOCIATION OF TECHNICAL STANDARDS – BRAZILIAN STANDARD. NBR 6355: Profiles of structural cold-formed steel – Standardization. Rio de Janeiro: [s.n.], 2012. 36 p.

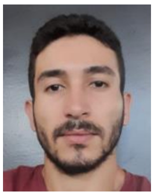
[16]. GERDAU. GERDAU Profiles – Metal construction: gauge table. Table of gauges. Available at: <https://www2.gerdau.com.br/>. Accessed on: August 24, 2022.

[17]. BRAZILIAN ASSOCIATION OF TECHNICAL STANDARDS – BRAZILIAN STANDARD. NBR 15980: Rolled steel shapes for structural use – Dimensions and tolerances. Rio de Janeiro: [s.n.], 2020. 26 p.

[18]. FISHER, J. M.; KLAYBER, L. A. Base plate and anchor rod design. Steel Design Guide 1. 2. ed. American Institute of Steel Construction. Chicago, 2006.

[19]. GERDAU. Connection for steel structures: a practical guide for structures with rolled profiles. Available at: <https://www2.gerdau.com.br/>. Accessed on: August 24, 2022.

Authors



João Victor Cordeiro Barros - Graduated in Civil Engineering (2023) from the Agreste Academic Center (CAA) and is a member of the Structural Engineering Research Group (GPEE) registered at CNPq and certified by the Federal University of Pernambuco (UFPE).

Douglas Mateus de Lima - Graduated in Civil Engineering (2010) from the Agreste Academic Center (CAA), holds a master's degree (2013) from the Graduate Program in Civil and Environmental Engineering (PPGECAM) and a doctorate (2018) from the Graduate Program in Civil Engineering (PPGEC), all from the Federal University of Pernambuco (UFPE). He is an Assistant-C3 professor at the Federal University of Pernambuco (UFPE), located at the Agreste Academic Center (CAA) and leader of the Structural Engineering Research Group (GPEE) registered at CNPq and certified by UFPE. He works on stability, dynamics and vibration control in tubular steel towers for horizontal axis wind turbines and also on numerical and experimental dynamic analysis of current structures in the Agreste region of Pernambuco.

Mateus Narcizo de Almeida Nunes – Graduated in Civil Engineering (2021) from the AESGA/FACIGA, he is a master's student (2022) in the Graduate Program in Civil and Environmental Engineering (PPGECAM) in the area of Structural and Materials Engineering at the Federal University of Pernambuco (UFPE) and is a member of the Research Group in Structural Engineering (GPEE) registered at CNPq and certified by the Federal University of Pernambuco (UFPE).