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Review Article

Stability Assessment of Steel Sway Portal Frames under Variations of Roof Pitch and Structural Stiffness

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Abstract

This study investigates the influence of the rafter pitch angle and the beam-to-column stiffness ratio on the buckling resistance of steel frame structures. The research focuses on steel portal frames subjected to compressive and combined loading conditions, aiming to evaluate the interaction between geometric configuration on the global stability behavior of the structural system. A parametric numerical analysis is performed by varying the roof inclination angle, and the relative stiffness of beams and columns. The critical buckling loads are determined by conducting linear buckling analysis for the parametric investigation of the steel frame. The critical loads are examined to identify the governing parameters affecting structural stability. In addition, geometrically nonlinear elastic analyses were carried out in order to investigate the post-buckling response of the structure. The equilibrium path of the frame was traced, revealing the existence of a stable secondary equilibrium path and demonstrating significant post-buckling strength. The results indicate that both the beam-to-column stiffness ratio significantly influences the buckling capacity of the frame, while the roof slope modifies the distribution of internal forces and the overall stability response. Increased joint flexibility generally reduces the critical load, whereas optimized stiffness ratios can enhance structural performance. The findings contribute to a better understanding of the stability and post-buckling behavior of steel portal frames and provide useful guidance for the design and optimization of lightweight steel structures.

Keywords: Buckling; critical buckling load; steel portal sway frames; stiffness coefficients; pitch angle; post-buckling behaviour

Introduction

The aim of this study is to examine the buckling behavior and overall stability response of steel sway portal frames. A thorough parametric analysis is conducted to assess the impact of essential structural parameters on the critical buckling load. The impact of the beam-to-column stiffness ratio and the roof inclination angle is analyzed comprehensively. The study seeks to elucidate the relationship between geometric properties and beam-to-column stiffness ratio, together with their impact on the overall stability of steel portal frame systems. A multitude of researchers have thoroughly investigated the phenomenon of buckling and the behavior of steel structures under conditions of instability. The examination of structural stability is a critical subject in structural mechanics and steel design, as buckling can profoundly influence the load-bearing capacity and safety of structural elements. Significant focus has been directed into the examination of steel structural members subjected to compressive loads, employing theoretical, experimental, and numerical methodologies.

The assessment of a column's buckling strength under axial compression can be performed using analytical or numerical techniques. The predominant analytical methods are the effective length and notional load methods [37], historically utilized by structural engineers for design, as noted by Connor [12], Chwalla [11], Petersen [32], [33], Pflüger [34], and Rubin [35]. These methods remain part of most contemporary structural design codes, such as Eurocode 3 [17,18] and LRFD [28]. These methods, designed for ease of hand calculations, rely on several simplifying assumptions that might significantly affect their accuracy. The significant advancements in recent years in computer hardware and engineering software enable the application of more precise numerical analysis algorithms, which can determine buckling strength through both linear and non-linear methods (pertaining to large displacements and/or material yielding) (Chen and White [5], White and Hajjar [41], [42], [43], Torkamani et al. [39], White and Hajjar [44], Chan [4], Torkamani and Sonmez [40], Kim et al. [24], Rubin [36]). Nonetheless, the vast majority of structural engineers continue to choose analytical methods, particularly during the basic design phase. Wood's work [45] formed the theoretical foundation of EC3. Cheong-Siat-Moy [6] investigated the k-factor dilemma for leaning columns, highlighting the reliance of buckling strength on both the rotational boundary conditions of the member and the overall behavior of the structural system. Bridge and Fraser [3] introduced an iterative method for assessing the effective length, which incorporates axial forces in the restraining members and hence accommodates negative values of rotational stiffness. Cheong-Siat-Moy [7] proposes a hypothetical lateral load method for assessing the strength of inclined columns. He [6] also investigated the k-factor dilemma for leaning columns and highlighted the reliance of buckling strength on both the rotational boundary conditions of the member and the overall behavior of the structural system. Aristizabal-Ochoa [1], [2] provided analytical formulas for assessing the effective length of columns in sway, non-sway, and partially sway frames, applicable to leaning columns. Hellesland and Bjorhovde [21] introduced a novel constraint demand factor that accounts for vertical and horizontal interactions in terms of member stability. They provided a method for evaluating the effective length, particularly in scenarios where significant alterations in column or beam stiffness arise, such as in the top or bottom stories of braced frames [22].

Kishi et al. [25] introduced an analytical formula for assessing the effective length of columns with semi-rigid connections in sway frames. Essa [19] proposed a design methodology for assessing the effective length of columns in unbraced multistory frames, taking into account various story drift angles. The buckling behavior of a metallic part is associated with the rotational stiffness of the neighboring members at the upper and lower joints. Consequently, the contribution of each member's rotational stiffness, contingent upon the boundary circumstances at its distal end, is ascertained utilizing the slope-deflection approach [37]. Cheong-Siat-Moy [8] focused on the conceptual comprehension of the behavior of leaning columns at the initiation of buckling. He further analyzed the behavior of columns within braced frames [9]. Cheong-Siat-Moy [10] proposes a universal formula for evaluating columns with lateral restraint that might range from negative infinity to positive infinity. Karamanos and Zissopoulou [23] analyzed the relevant stipulations of EC3 and LRFD concerning sway frames. Gantes and Mageirou [20], as well as Mageirou and Gantes [29], developed enhanced stiffness distribution parameters for assessing effective buckling lengths in multi-story frames exhibiting varying degrees of sway capability. They [30, 31] also examined the buckling behavior of multi-story sway, non-sway, and partially-sway frames with semi-rigid connections. Consequently, they presented analytical formulas and diagrams for assessing effective buckling lengths in multi-story frames with semi-rigid connections. Kounadis et al. [26] examine the buckling strength of a member using non-linear approaches, considering high displacements and/or material yielding, through both analytical and numerical methods. Thanopoulos et al. [41] examine strategies for the design of bridge plate girders to mitigate lateral torsional buckling. Their analysis juxtaposes the outcomes of the critical load assessment with those generated from the design load in accordance with Eurocode stipulations.

Steel Structure under Consideration

Initially, the analysis and design of a steel portal frame industrial shed with plan dimensions of 10 m × 55 m is carried out using SAP2000 software [42]. The steel frames are spaced at 5 m intervals and consist of columns with a height of 5 m (h = 5 m) and a bay width of 10 m (L = 10 m). The steel grade used is S235. Each frame is assumed to be sway, with fixed supports at both columns' ends. The frame is designed to satisfy the provisions of Eurocodes 0 and 3 [8, 9] for the typical loading conditions specified in Eurocodes 1 [43, 44] and 8 [45]. More specifically, the nominal loads acting on the frame are as follows (Figure 1).

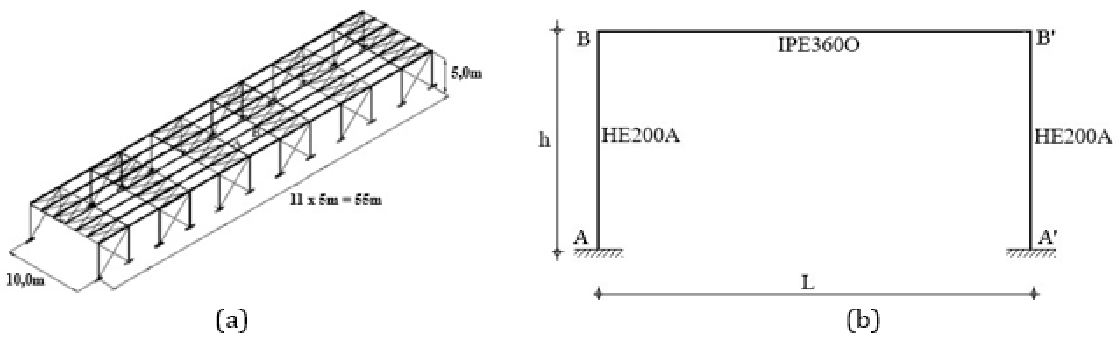

Figure 1: a) Steel structural industrial building, b) Steel portal frame.

The parameters for the design of the steel frame are defined as follows. The structural system consists of a planar portal frame subjected to the relevant permanent and variable actions in accordance with Eurocode provisions. The columns are made of HE200A sections, while the rafters are initially selected as IPE3600 profiles. Material properties correspond to structural steel with standard elastic modulus and yield strength as specified in Eurocode 3. Geometric and boundary conditions, including member lengths, frame height, and support constraints, are also taken into account in the analysis model. More specifically, the nominal loads are given below.

Panels load: 0,12 kN/m^2

Live load (non-accessible roof): 5 kN/m

Snow load: 1,6 kN/m

Wind load: wind pressure on the windward column: 2,5 kN/m

Wind load: wind pressure, suction on the leeward column: 1,5 kN/m

Wind load: wind pressure, suction on the rafter: 1,0 kN/m

Temperature variation ΔT: ±20°C

Ground acceleration: 0,24 g

Response spectrum: EC8 code provisions

Ductility class: low

Behaviour factor: 1,15

Capacity design: yes

Connections classification: continuous

After completing the strength and serviceability checks in accordance with the provisions of Eurocode 3 [8,9], the final selection of the structural members was carried out. The adopted structural system consists of HE200A sections for the columns and IPE360O sections for the rafters. This choice ensures that the frame satisfies all relevant design requirements under the considered loading conditions [43, 44, 45], while maintaining adequate structural performance in terms of both resistance and stiffness. Afterwards, once the design of the frame had been completed, a linearized buckling analysis [42] of the portal frame was performed, resulting in a critical buckling load for the first eigenmode equal to 2663 kN (Figure 2).

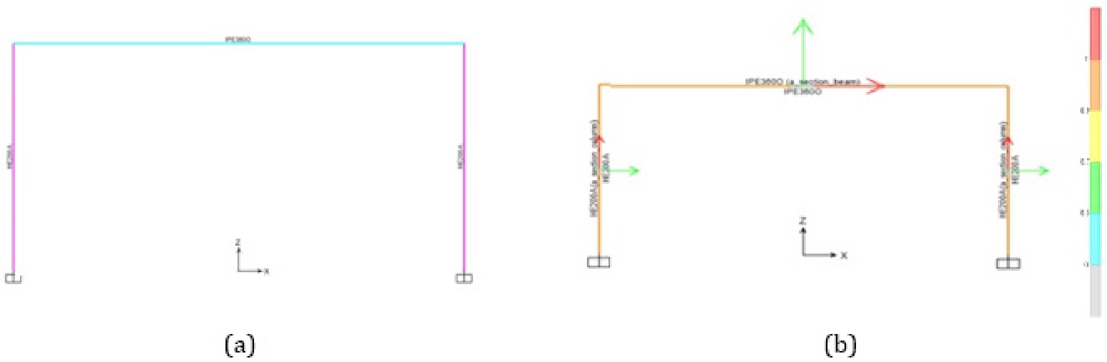

Figure 2: a) Steel structural frame, b) Utilization factors of the sections for the steel frame.

The influence on the column buckling resistance (critical buckling load) of the aforementioned steel frame is investigated with respect to the following parameters:

a. Beam-to-column stiffness ratio: variation of the beam (rafter) cross-section while keeping the column section constant (HE200A).

b. Beam-to-column stiffness ratio: variation of the column cross-section while keeping the beam section constant (IPE360O).

c. Rafter inclination angle.

d. Semi-rigid behavior of the beam–column joint.

The parameters kept constant throughout the parametric study are the frame height (h) and span length (L), with h = 5 m and L = 10 m.

Investigation of the Influence of the Rafter Inclination on the Buckling Resistance of the Steel Frame.

Investigation of the influence of the rafter inclination on the buckling resistance of the steel frame. Initial steel structure under consideration.

The variation of the critical buckling load is also investigated for pitched (inclined) frames with different beam angles. The beam inclination angles range from zero to thirty-five degrees (0° to 35°). In typical roof structures under mild winter conditions, the roof slope does not usually exceed ten degrees. However, larger inclination angles are also considered in order to obtain a clearer understanding of the relationship between beam slope and the critical buckling load of the frame columns. The normalization of the frame's critical load was carried out, as in previous cases, using the Euler buckling load of a pinned-pinned HE200A column. Linearized buckling analyses were performed for the following structural configurations:

a) a steel frame with fixed column and beam sections (HE200A and IPE360), b) steel frames with a constant column section and varying beam sections, c) steel frames with a constant beam section and varying column sections (Tables 1-4, Figures 3-6).

Table 1: Investigation of the influence of the rafter inclination (0° έως 35°) on the buckling resistance of the steel frame.

slope of the steel frame rafter φ (o)	P_{cr} (kN)	$P_{cr}/P_{cr,E}$
0	26,625,891	24,039
5	26,615,891	24,030
10	26,578,602	23,996
15	26,517,994	23,942
20	26,430,232	23,862
25	26,312,159	23,756
30	26,159,173	23,618
35	25,964,913	23,442

Table 2: Investigation of the influence of the rafter inclination (0o έως 5o) on the buckling resistance of the steel frame

slope of the steel frame rafter φ (o)	P_{cr} (kN)	$P_{cr}/P_{cr,E}$
0	26,625,891	240,391
1	26,625,423	240,387
2	26,624,019	240,374
3	26,621,679	240,353
4	26,618,396	240,324
5	266,158,913	240,301

Table 3: Investigation of the influence of the rafter inclination (15o έως 20o) on the buckling resistance of the steel frame

slope of the steel frame rafter φ (o)	P_{cr} (kN)	$P_{cr}/P_{cr,E}$
15	26,517,994	23,942
16	26,502,713	23,928
17	26,486,321	23,913
18	26,468,788	23,897
19	26,450,119	23,880
20	26,430,232	23,862

Table 4: Investigation of the influence of the rafter inclination (30o έως 35o) on the buckling resistance of the steel frame

slope of the steel frame rafter φ (o)	P_{cr} (kN)	$P_{cr}/P_{cr,E}$
30	26,159,173	23,618
31	26,123,856	23,586
32	26,086,857	23,552
33	26,048,073	23,517
34	26,007,454	23,481
35	25,964,913	23,442

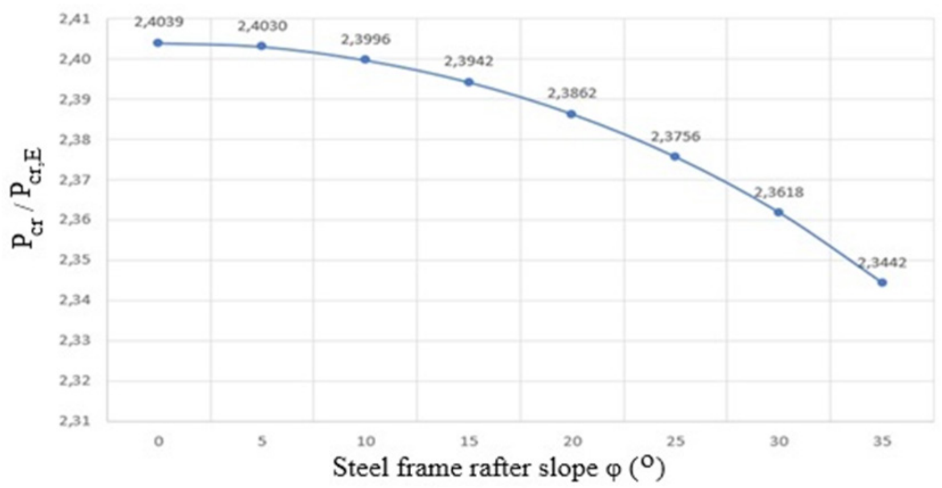

Figure 3: Investigation of the influence of the rafter inclination (0o έως 35o) on the buckling resistance of the steel frame.

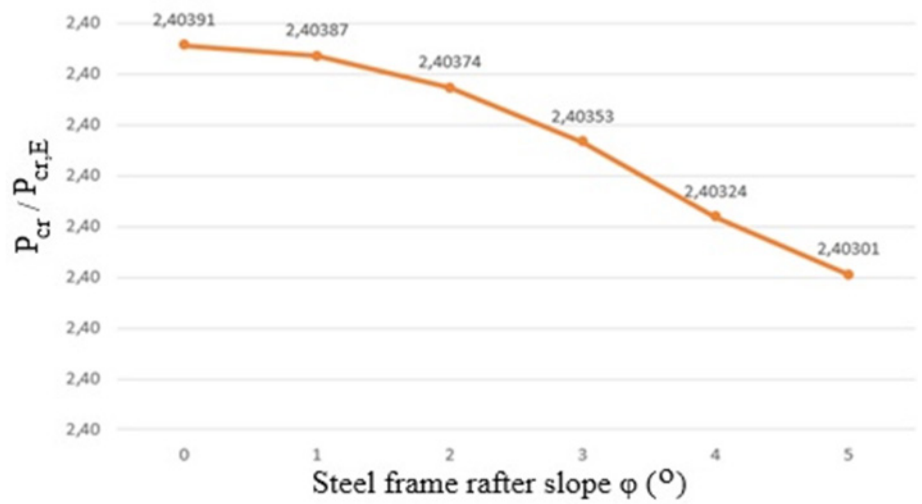

Figure 4: Investigation of the influence of the rafter inclination (0o έως 5o) on the buckling resistance of the steel frame.

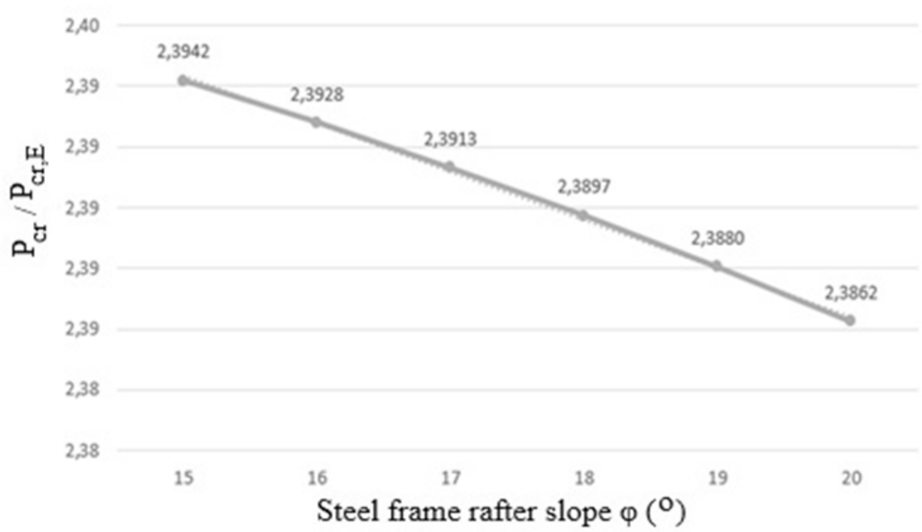

Figure 5: Investigation of the influence of the rafter inclination (15o έως 20o) on the buckling resistance of the steel frame.

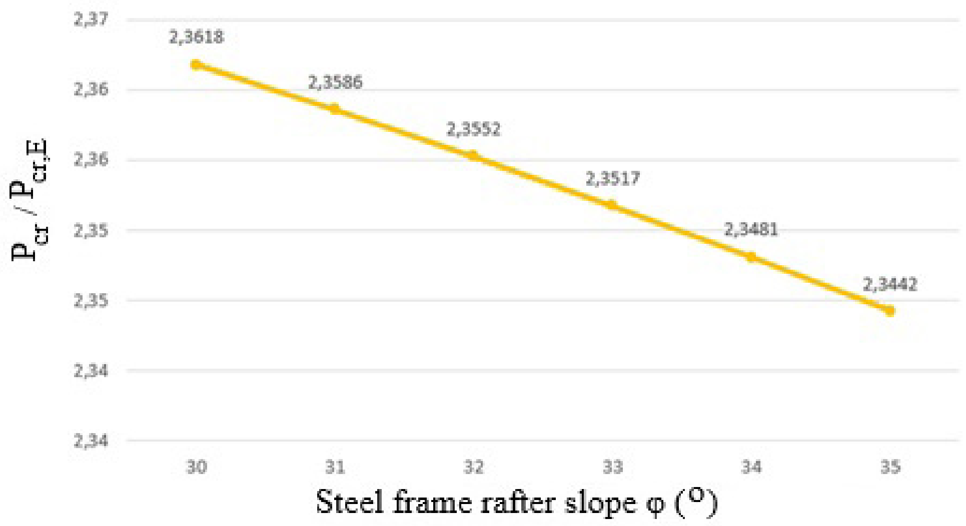

Figure 6: Investigation of the influence of the rafter inclination (30o έως 35o) on the buckling resistance of the steel frame

It is observed that an increase in the beam inclination leads to a reduction in the critical buckling load. However, from the parametric analyses, the overall decrease in the critical load for inclinations ranging from 0° to 35° is only about 2%. Therefore, in this particular case, the beam inclination does not significantly affect the buckling behavior of the frame. The variation of the critical buckling load for different ranges of beam angles is as follows: a) from 0° to 5°, the reduction is 0.04%, b) from 15° to 20°, the reduction is 0.33%,

while for inclinations from 30° to 35°, a reduction of 0.75% is observed. It is further noted that the decrease in the critical buckling load becomes more pronounced at larger beam inclinations.

Investigation of the influence of the rafter inclination on the buckling resistance of the steel frame. Variation of the column cross-section while keeping the beam section constant

With the beam section kept constant as IPE360 (I2 constant), frame structures with different beam-to-column stiffness ratios were investigated:

a. I2/I1= 0,25 (column cross-section HE450B)

b. I2/I1=0,50 (column cross-section HE340B)

c. I2/I1=0,75 (column cross-section HE300B)

d. I2/I1=1,00 (column cross-section HE300A)

Figure 7: Critical buckling load of steel frame with rafter slope a) 5o, b) 35o

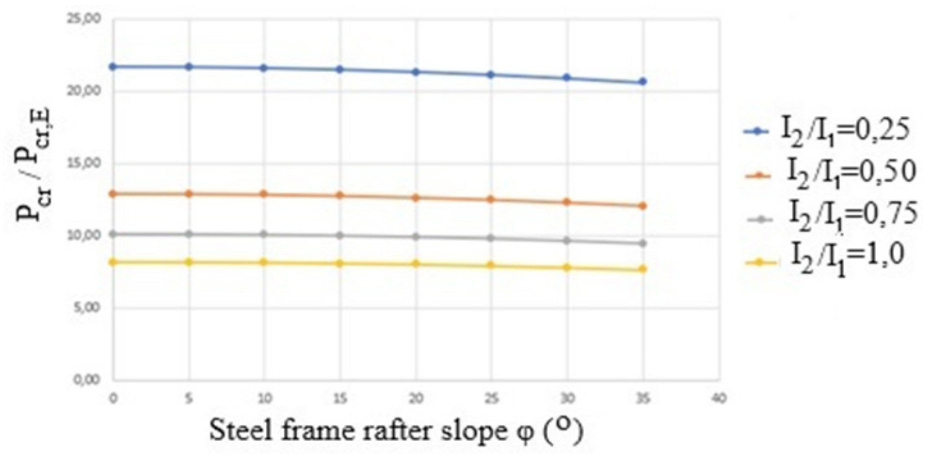

Figure 8: Normalized buckling load $P_{cr}/P_{cr,E}$, for different beam inclinations and beam-to-column stiffness ratios

Table 5: Critical buckling load P_{cr} (kN) for different beam inclinations and beam-to-column stiffness ratios

slope of the steel frame rafter φ (o)	P_{cr} (kN) (I2/I1=0,25)	P_{cr} (kN) (I2/I1=0,5)	P_{cr} (kN) (I2/I1=0,75)	P_{cr} (kN) (I2/I1=1,0)
0	2,398,594,537	1,424,523,157	118,824,875	902,895,881
5	239,615,399	1,422,733,491	1,117,425,725	901,840,408
10	2,388,830,652	1,417,351,727	1,113,212,137	898,657,293
15	2,376,617,153	1,408,337,714	1,106,133,991	893,294,866
20	2,359,501,166	1,395,623,920	109,610,637	885,664,916
25	2,337,468,544	1,379,116,909	1,083,009,357	875,640,919
30	2,310,509,433	1,358,700,907	1,066,688,967	863,056,368
35	2,278,615,187	1,334,234,372	1,046,951,977	847,697,563

Table 6: Normalized buckling load $P_{cr}/P_{cr,E}$, for different beam inclinations and beam-to-column stiffness ratios

slope of the steel frame rafter φ (o)	$\mathbf{P_{cr}/P_{cr,l}}$ **(I2/I1=0,25)**	$\mathbf{P_{cr}/P_{cr,E}}$ **(I2/I1=0,5)**	$\mathbf{P_{cr}/P_{cr,E}}$ **(I2/I1=0,75)**	$\mathbf{P_{cr}/P_{cr,E}}$ **(I2/I1=1,0)**
0	2,165,566	1,286,127	1,010,128	815,177
5	2,163,362	1,284,512	1,008,865	814,225
10	2,156,750	1,279,653	1,005,061	811,351
15	2,145,723	1,271,515	998,671	806,509
20	2,130,270	1,260,036	989,617	799,621
25	2,110,378	1,245,133	977,793	790,570
30	2,086,038	1,226,700	963,058	779,208
35	2,057,243	1,204,610	945,238	765,342

It is observed that the rate of reduction in the buckling load is similar (6,34%, 6,42%, and 6,11%, respectively) for beam-to-column stiffness ratios (I_2/I_1) of 0,5 - 0,75, and 1,0. It is therefore concluded that the beam inclination does not significantly influence the value of the critical buckling load for stiffness ratios within the range discussed above.

For a stiffness ratio of 0,25, the percentage reduction in the critical buckling load is approximately 5%. This occurs because, as previously noted, a small column cross-section relative to the beam section results in the beam acting as a fixed support. Consequently, the column behaves as a fixed-fixed member, in which case the influence of the beam inclination on the structural response is less pronounced than in any other configuration.

Investigation of the influence of the rafter inclination on the buckling resistance of the steel frame. Variation of the beam (rafter) cross-section while keeping the column section constant.

With the column section kept constant, HE200A (I1: constant), frame structures were investigated for beam-to-column stiffness ratios:

a. I2/I1= 0,25 (beam cross-section IPE160)

b. I2/I1=0,50 (beam cross-section IPE200)

c. I2/I1=0,75 (beam cross-section IPE220)

d. I2/I1=1,00 (beam cross-section HE300A)

e. I2/I1=2,00 (beam cross-section IPE270R)

Table 7: Critical buckling load P_{cr} (kN) for different beam inclinations and beam-to-column stiffness.

slope of the steel frame rafter φ (o)	P_{cr} (kN) (I2/I1=0,25)	P_{cr} (kN) (I2/I1=0,5)	P_{cr} (kN) (I2/I1=0,75)	P_{cr} (kN) (I2/I1=1,0)	P_{cr} (kN) (I2/I1=2,0)
0	114,452,471	14,851,153	168,114,023	188,308,256	22,511,317
5	114,328,794	14,831,521	167,894,571	188,080,165	224,909,393
10	113,957,921	14,772,511	167,233,960	187,392,459	224,293,222
15	113,340,193	14,673,759	166,125,158	186,234,503	223,249,653
20	112,476,154	14,534,651	164,556,251	184,588,141	221,752,682
25	111,366,667	14,354,340	162,510,439	182,427,345	219,763,986
30	110,013,171	14,131,781	159,966,225	179,717,900	217,231,093
35	108,417,457	13,865,695	156,896,661	176,415,919	214,083,706

Table 8: Normalized buckling load $P_{cr}/P_{cr,E}$, for different beam inclinations and beam-to-column stiffness ratios

slope of the steel frame rafter φ (o)	$P_{cr}/P_{cr,E}$ (I2/I1=0,25)	$P_{cr}/P_{cr,E}$ (I2/I1=0,5)	$P_{cr}/P_{cr,E}$ (I2/I1=0,75)	$P_{cr}/P_{cr,E}$ (I2/I1=1,0)	$P_{cr}/P_{cr,E}$ (I2/I1=2,0)
0	103,333	134,083	151,781	170,014	203,243
5	103,221	133,906	151,583	169,808	203,059
10	102,887	133,373	150,987	169,187	202,503
15	102,329	132,482	149,986	168,141	201,560
20	101,549	131,226	148,569	166,655	200,209
25	100,547	129,598	146,722	164,704	198,413
30	0,99325	127,588	144,425	162,258	196,127
35	0,97884	125,186	141,654	159,277	193,285

Table 9: Load – displacement of node 3 - elastic nonlinear analysis.

P (kN)	W (cm)	P (kN)	W (cm)	P (kN)	W (cm)	P (kN)	W (cm)
35,549,010	0,00344	681,417,890	0,08729	138,681,240	0,26231	2,139,725	0,73811
71,110,560	0,00693	717,732,410	0,09354	142,530,386	0,27651	2,188,818	0,79118
106,685,000	0,01047	754,109,200	0,10001	146,401,164	0,29152	2,239,533	0,85019
142,272,670	0,01405	827,063,680	0,11368	150,295,429	0,30738	2,292,156	0,91621
178,075,580	0,01842	863,649,110	0,12091	154,215,256	0,32417	2,347,087	0,9907
213,690,830	0,02211	900,312,330	0,12843	158,162,971	0,34199	2,404,714	107,509
249,320,480	0,02584	93,705,802	0,13625	162,141,192	0,36093	2,465,657	117,167
285,278,250	0,03078	97,389,123	0,14439	166,152,874	0,38109	2530,69	128,328
320,937,940	0,03463	101,081,744	0,15287	170,201,366	0,40259	2,600,842	141,371
356,919,370	0,03965	104,784,260	0,16171	174,303,596	0,42607	2,677,523	156,818
392,610,920	0,04361	108,497,316	0,17094	178,439,429	0,45078	2,762,714	175,400
428,711,280	0,04907	112,221,617	0,18058	182,625,512	0,47733	2,859,257	198,166
464,672,010	0,05402	115,957,929	0,19066	186,867,681	0,50594	2,971,508	226,735

500,673,660	0,05911	119,707,093	0,2012	1,911,727	0,53684	3,106,317	263,653
536,721,120	0,06438	123,470,027	0,21226	1,955,485	0,57034	3,275,369	313,285
572,816,730	0,06982	127,247,740	0,22385	2,000,042	0,60678	3,500,471	383,833
608,962,950	0,07545	131,041,342	0,23603	2045,51	0,64655	3,834,071	495,276
645,162,400	0,08127	134,852,058	0,24883	2,092,019	0,69013	4625,72	784,583

It is observed that for a beam-to-column stiffness ratio (I_2/I_1) of 0.25, the column approaches cantilever-like behaviour. For this reason, the percentage reduction in the critical buckling load is smaller compared to the other cases. It is also observed that as the beam inclination increases, the critical buckling load decreases accordingly. Similarly to the previously discussed cases, the reduction is more pronounced for larger beam angles.

In conclusion, the beam inclination has only a minor influence on the buckling resistance of the structure. This behaviour is expected for beam inclination ranges similar to those investigated in this study.

Elastic Nonlinear Behaviour of the Steel Frame

An elastic nonlinear analysis of the steel frame from Section 1 is performed. An initial geometric imperfection equal to 20% of the first buckling mode is introduced, in accordance with the provisions of Eurocode 3. The critical buckling load is determined from a linearized buckling analysis and, as previously stated in Section 1, is equal to 2663 kN. Following the elastic nonlinear analysis of the steel frame, the equilibrium path is plotted for node 13, which exhibits the maximum displacement. The results are presented both in tabular form and as a graphical representation (Table 9, Figure 10).

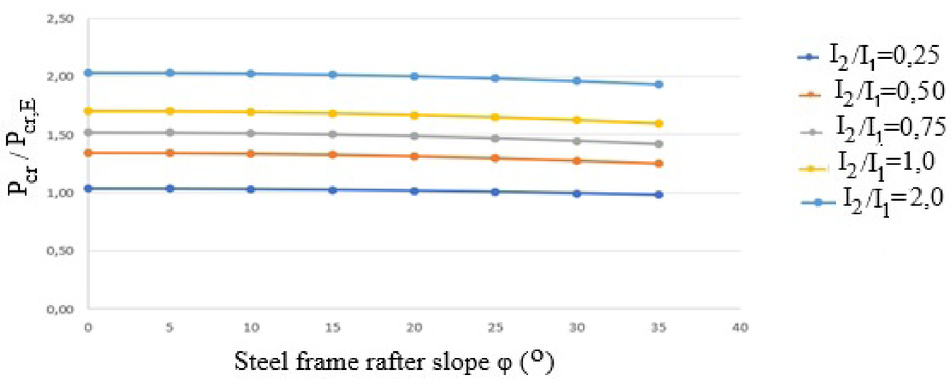

Figure 9: Normalized buckling load $P_{cr}/P_{cr,E}$, for different beam inclinations and beam-to-column stiffness ratios

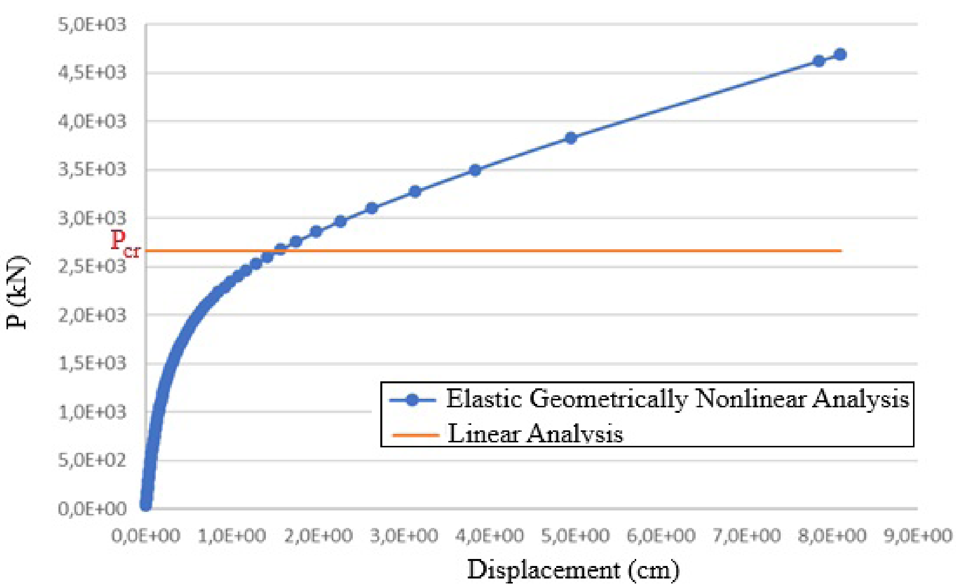

Figure 10: Comparison of elastic nonlinear analysis results and elastic critical buckling load.

Conclusions

The influence of the roof slope angle of a pitched gable frame and the beam-to-column stiffness ratio on the buckling resistance of a planar steel frame was investigated in detail. The considered structural system represents a typical component of an industrial steel shed. Initially, the frame was designed for the relevant permanent and variable actions in accordance with Eurocodes 0, 1, 3, and 8, ensuring that all basic strength and serviceability requirements were satisfied prior to the parametric investigation.

Subsequently, a systematic study was conducted on the effect of the roof pitch angle of the gable frame on the critical buckling load of the structure. It was observed that an increase in the beam inclination leads to a reduction in the critical load, with more pronounced decreases occurring at larger inclination angles. Nevertheless, when considering a relatively wide range of roof slopes from 0° to 35°, the overall variation in the critical buckling load was found to remain limited, not exceedingly approximately 2%. As a result, the roof slope does not appear to play a dominant role in the global buckling response of the frame. In addition, the combined influence of beam inclination and beam-to-column stiffness ratio was examined, confirming that the sensitivity of the buckling resistance to changes in beam slope remains generally low across the investigated configurations.

The reduction rate in the buckling load is consistent at 6.34%, 6.42%, and 6.11% for beam-to-column stiffness ratios (I_2/I_1) of 0.5, 0.75, and 1.0, respectively. Consequently, it may be inferred that beam tilt does not substantially affect the critical buckling load for stiffness ratios within this range. With a stiffness ratio of 0.25, the critical buckling load experiences an estimated drop of 5%. This behavior arises because, as previously mentioned, the relatively diminutive column cross-section in comparison to the beam section causes the beam to function effectively as a stiff constraint. As a result, the column functions akin to a fixed-fixed member, whereby the impact of beam inclination on the total structural reaction is diminished and becomes less relevant compared to other designs. In conclusion, the beam inclination exerts a negligible effect on the buckling resistance of the structure. This behavior is anticipated for beam inclination ranges comparable to those examined in this study.

Finally, an elastic nonlinear buckling analysis of the frame was performed in order to further explore the structural behavior beyond the onset of instability. The obtained results indicate that the secondary (post-buckling) equilibrium path corresponds to a stable equilibrium configuration, suggesting that the structure is capable of sustaining additional load even after reaching the critical buckling point. This behavior confirms the presence of a stable post-buckling response and, consequently, demonstrates that the examined frame exhibits a measurable level of post-buckling strength and structural reserve capacity.

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