Effectiveness of Analysis Procedures on Buckling Loads of Steel Portal Frames

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Abstract: ***In this paper, different finite element models have been adopted to investigate the effectiveness of analysis procedures on the critical buckling loads of steel portal frame. In the linear approach where most analysis softwares adopt, the members imperfection are neglected, this can be leads to overestimate frame buckling strength. Therefore this paper aim to compare the critical loads obtained by the linear eigenvalue buckling analysis with corresponding value determined by the non-linear Riks method where the members imperfection are included. Different beam, column and loading nature has been considered in the finite element simulation. The results indicated that, the critical buckling loads obtained by the linear approach could give an overestimate buckling strength by 18% for the steel portal frame loaded with line load on its beam. A critical load reduction factor has been developed, for frames that are imperfection sensitive, with correlation coefficient of 0.907.***

Keywords: Buckling Loads, Steel portal frame, finite element modelling, imperfection sensitivity, Riks method.

1. Introduction

Structure stability is an important parameter in designing structures. In many instances, buckling is the primary consideration in the design of various structural configurations, where the buckling load could be remarkably lower than the load cause yielding in some members. Portal frame is an example of these structures. A portal frames similar to that shown in Figure 1 is usually adopted in single-story buildings for industrial, warehousing or other purposes.

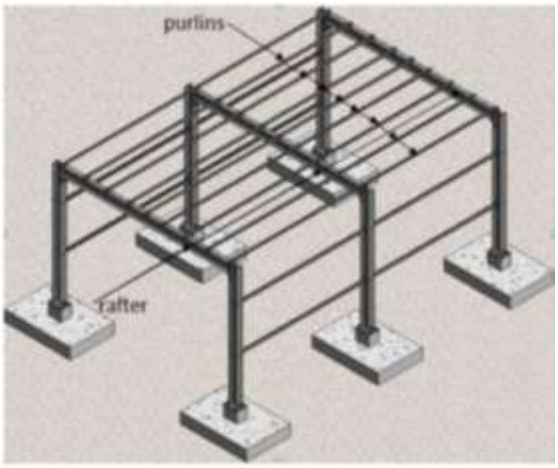

Figure 1: Portal frame [1].

In most general engineering software, the buckling analysis is executed by linear Eigenvalue buckling analysis. This paper aims to compare the linear buckling load with the buckling load obtained from a more sophisticated nonlinear analysis.
In this work, Abaqus software has been adopted to execute both linear and non-linear analysis, where the Eigenvalue buckling analysis is used as an example for the adopted analysis in routine design work to determine linear elastic buckling load while the Riks method is adopted for more sophisticated nonlinear buckling analysis, where the member imperfection can be included.

2. Review of Literature

2.1. Stability

Structure stability should be well-understood to get more economical use of the material, where in almost all constructions, when demanding the use of higher strength members with the lighter weight that can be associated; the consideration of structure stability must play a crucial role in design. Increased strength and increased slenderness invariably lead to problems with instability [2].

2.2. Linear analysis

The critical buckling load is the main criterion to measure any structure stability. The critical load is the load that corresponding to a condition in which a perturbation of the deformation status does not interrupt the equilibrium between internal and external forces [3].

In the stiff structures, the estimation of critical buckling loads can be done by executing linear eigenvalue buckling analysis. Stiff structures are those structures carry their designed loads mainly by axial force or membrane force, rather than bending action. Their response involves very small deformation before buckling [4]. Although the responses of a structure are nonlinear before the collapse, an eigenvalue buckling analysis will provide a useful estimation of the collapse mode shapes [4].

The searching in the eigenvalue buckling problems is for the loads that make the stiffness matrix become singular [5]. So that the problem of

$K^{MN}\ V^{M} = 0$ has a non-trivial solution, where

K^{MN} is the tangent stiffness matrix, when the loads are applied,

V^{M} is a non-trivial displacement solutions.

The linear buckling analysis is widely used in almost all commercial software because the linear approaches are simple and fast to solve. The problems in the real world are only approximated to be linear; the more accurate results can be gain with nonlinear analysis, and that is the main interest of this research.

2.3. Non-linear analysis

In many instances, the system behavior showed a reasonable idealization after the linear analysis. However, in other cases, the results may present an unrealistic approximation of the response. Therefore, the analysis type (linear or nonlinear) depends on the main goal of the analysis and the system's response errors that may be accepted. In some cases, the only option is the nonlinear analysis for the designer as well as the analyst [6].

The nonlinear geometry problems sometimes involve buckling or collapse response, in which the load-displacement response showed a negative stiffness, in that case and to stay in equilibrium the structure must release some strain energy. Often it is necessary to obtain nonlinear static equilibrium solutions for the unstable problems, where the load-displacement response shows the behavior sketched in Figure 2, that is, when the load and/or the displacement may decrease as the solution evolves. The modified Riks method is an algorithm that allows an effective solution of such cases [4].

The response in the post-buckled is unstable in many cases; thus, the collapse loads will strongly depend on original geometric imperfections, and that known as imperfection sensitivity; in that scenario, the actual failure load could be remarkably lower than the buckling load estimated by the eigenvalue buckling analysis. However, the eigenvalue buckling analysis provide a non-conservative estimation of the structural load carrying capacity even if the pre-buckling responses are stiff and linear elastic, a nonlinear load-displacement response analysis for the imperfect structures are generally recommended to follow the eigenvalue buckling analysis, that is mainly if the structures are imperfection sensitive [4].

In the Riks method the load magnitude is used as an additional unknown; where it solves simultaneously for both loads and displacements. Thus, another value must be add to measure the solution progress; Abaqus uses the arc length, (l), along with the static equilibrium path in load-displacement space as shown in Figure 3. This approach provides solutions regardless of whether the response is stable or unstable.

The Riks method also can be used to solve post-buckling problems, with both stable and unstable response. However, the exact post-buckling problems cannot be analyzed in a simple direct way due to a discontinuous response showed by the structure at the point of buckling, and to analyze a post-buckling problem therefore and instead of bifurcation problem the response must be turned into a continuous response. This effect can be established by introducing some initial imperfection into the perfect geometry. Thus in the buckling mode, there is some response before reaching the critical load [4].

The geometric imperfection is based on eigenvalue buckling analyses, where it captures one of the buckling mode shapes. However, the lowest magnitude of the buckling modes is assumed to provide the most critical imperfections, therefore the lower modes usually are scaled and assigned to the perfect geometry. The imperfection magnitude should be realistically chosen. The manufacturing tolerances may be chosen to determine the imperfection size for example. The magnitude often is chosen as a percent of dimension for a relevant structure such as a beam cross-section or a shell thickness [4]. In this study and according to AISC recommendations for the maximum fabrication tolerance, a geometric imperfection of ($L/1000$) is adopted [7].

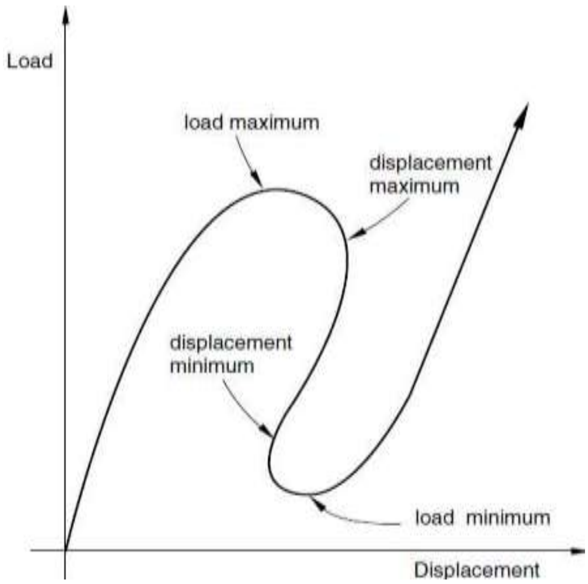

Figure 2: Typical unstable static response [4]

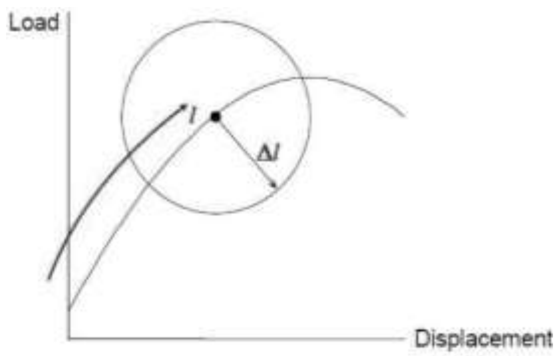

Figure 3: Arc length and arc length increment [4].

3. Finite element modeling

The structure was simulated in a two-dimensional plane. Therefore, it has been enforced to buckle in the plane where the moment is about its major axes as shown in Figure 4 and figure 5. Due to existing of purlins and bracings shown in Figure 1, designers usually concern with sway and non-sway buckling modes that produce bending moments about major axes of rafters and columns [8].

3.1. Element Type and size

In this study, the plane frame element of Figure 6 has been adopted, this element can be assigned with designated cross section, and furthermore it can give the prospective deformed buckling mode shape.

The mesh size has been adopted after performing multiple analysis (which has not been covered in this study). However, the chosen mesh size can give accurate results and sense the required member imperfection with best time consuming, a mesh size of 0.5 meter has been adopted for all case studies.

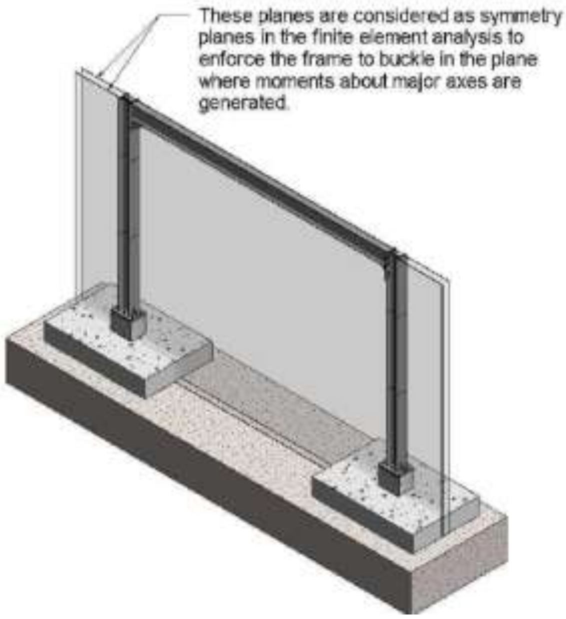

Figure 4: A typical internal frame that considered in finite element simulation of this study [1].

Figure 5: Modeling of portal frame in Abaqus.

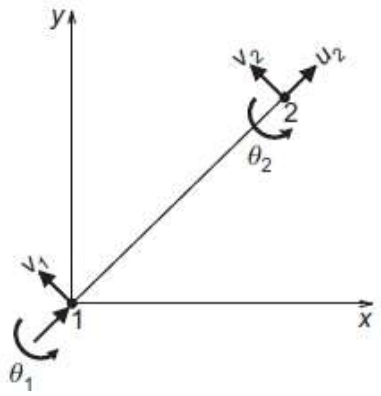

Figure 6: Plane frame element adopted in the simulation of the beam and columns.

3.1. Geometric properties

The frame bay length and heights were changing as each individual case study indicted. However, the assigned column profile was HEB200, and the assigned beam profile was varied between IPE200, IPE220 and IPE270.

Figure 7: Frame meshing

3.2. Material property

As stated earlier this paper aim to compare the buckling load obtained from linear and nonlinear analysis i.e. measuring the imperfection sensitivity for steel portal frame, therefore, the material behavior was assumed to be limited in the elastic zone with modulus of elasticity and passion ratio equal to 200,000 MPa and 0.3 respectively.

3.3. Modeling of initial imperfection

Initial local and overall geometric imperfections can be predicted from finite element models by conducting an eigenvalue buckling analysis to obtain the worst cases of local and overall buckling modes. These local and overall buckling modes can be then factored by measured magnitudes in the tests. Superposition can be used to predict final combined local and overall buckling modes. The resulting combined buckling modes can be then added to the initial coordinates of the structural member. The final coordinates can be used in any subsequent nonlinear analysis. In this study and according to AISC recommendations for the maximum fabrication tolerance, the nonlinear Riks analysis was performed after assigning a geometric imperfection of $(L/1000)$ form the buckling mode shape obtained from the linear eigenvalue buckling analysis. The first cases were simulated with imperfection based on sidesway uninhibited buckling mode shape as shown Figure 8, and the second cases were then simulated with imperfection based on sidesway inhibited buckling mode shape as shown in Figure 9.

Figure 8: Sidesway uninhibited buckling mode shape.

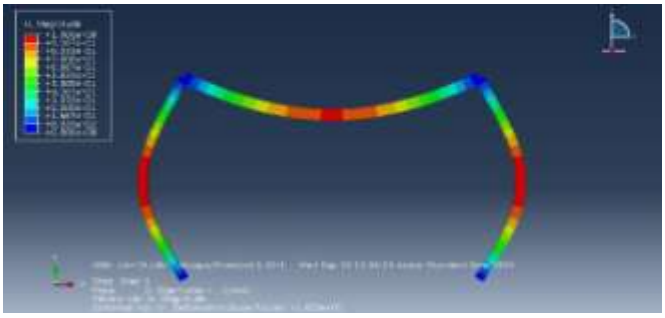

Figure 9: Sidesway inhibited buckling mode shape.

3.4. Loading and Boundary conditions

The columns bases were constrained against movement in all directions i.e. hinged supports has been assigned for all cases. After that and to measure the frame sensitivity to imperfection with deferent load nature, two kinds of load where applied, the first type was a concentrated loads applied at columns top as shown in Figure 10 and the second type was a line load applied at the beam as shown in Figure 11.

Figure 10: Applying concentrated loads at columns.

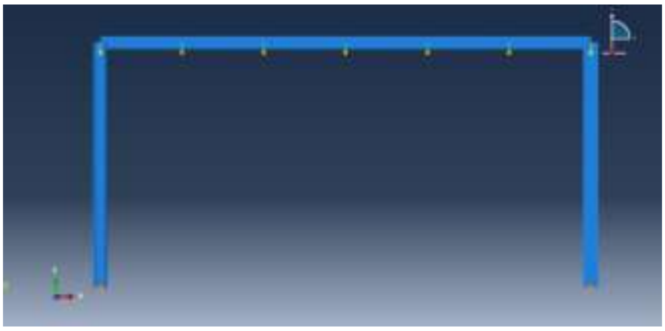

Figure 11: Applying uniformly distributed load at the beam.

4. Model Validation

The proposed finite element model was validated by comparing its critical buckling load with the load obtained by the traditional analytical approach.

In the finite element model, the buckling loads for the two buckling mode shape (sidesway inhibited and sidesway uninhibited) were obtained by applying concentrated loads at columns top. However, in the traditional approach, the buckling loads were obtained from Euler equation, as shown in equation (1) below after deriving the effective length factor (k) for the two buckling mode shape from equation (2) for sidesway inhibited and equation (3) for sidesway uninhibited [9].

$$p_{cr} = \frac{\pi^2 EI}{(kl)^2} \qquad \text{...(1)}$$

$$\frac{G_A G_B}{4}(\pi/K)^2 + \left(\frac{G_A + G_B}{2}\right)\left(1 - \frac{\pi/k}{\tan(\pi/k)}\right) + \frac{2\tan(\pi/2k)}{(\pi/k)} - 1 = 0 \qquad \text{...(2)}$$

$$\frac{G_A G_B (\pi/k)^2 - 36}{6(G_A + G_B)} - \frac{\pi/k}{tan\,(\pi/k)} = 0 \qquad \text{...(3)}$$

Where

$$G = \frac{\Sigma(E_c I_c / K_c)}{\Sigma(E_b I_b / K_b)} \qquad \text{...(4)}$$

Equation (4) has been solved for different values of *GA* and *GB* using the goal seek Excel formula. The subscripts A and B refer to the joints at the ends of the considered column. Where the hinge support can be interpreted as a very flexible beam, therefore the ratio approaches to a very high value, a value of 10 is usually adopted [10].

Figure 12 and Figure 13 presents the results comparison between the two approaches, were the proposed finite element model showed a good agreement with the traditional approach.

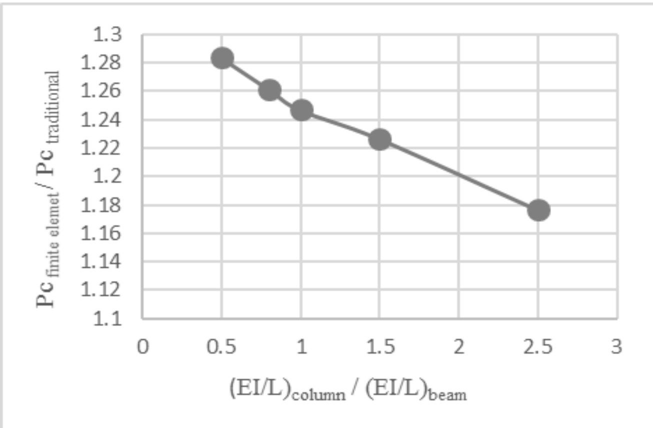

Figure 12: Validation results for sidesway uninhibited frame.

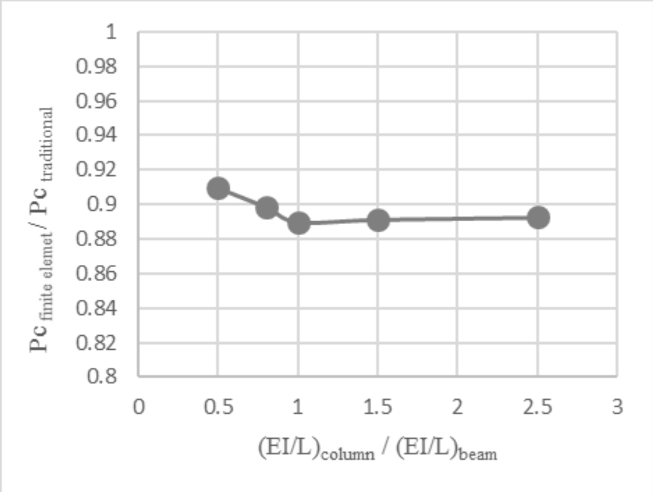

Figure 13: Validation results for sidesway inhibited frame.

5. Case studies

As stated earlier different load nature were applied at the steel portal frame i.e. concentrated load applied at columns top and uniformly distributed load applied at the beam. Deferent column to beam stiffness has been considered, the imperfection sensitivity for each case has been determined based on the ratio of buckling load obtained by the eigenvalue buckling analysis to the buckling load obtained by the non-linear Riks method analysis. The analysis results for each case are presented as follow.

5.1. Sidesway uninhibited frame loaded with line load

In this case, three types of frames has been simulated as follow;

- Frame 1: consist of HEB200 columns and IPE200 beam.
- Frame 2: consist of HEB200 columns and IPE270 beam.
- Frame 3: consist of HEB200 columns and IPE220 beam.

The results indicated that; as the slenderness ratio increased the frames will be more sensitive to imperfection as shown in Figure 14, where the critical load obtained by the nonlinear Riks method could be less by 18% than the critical load obtained by the linear Eigenvalue approach, summary of analysis results are presented in Table 1.

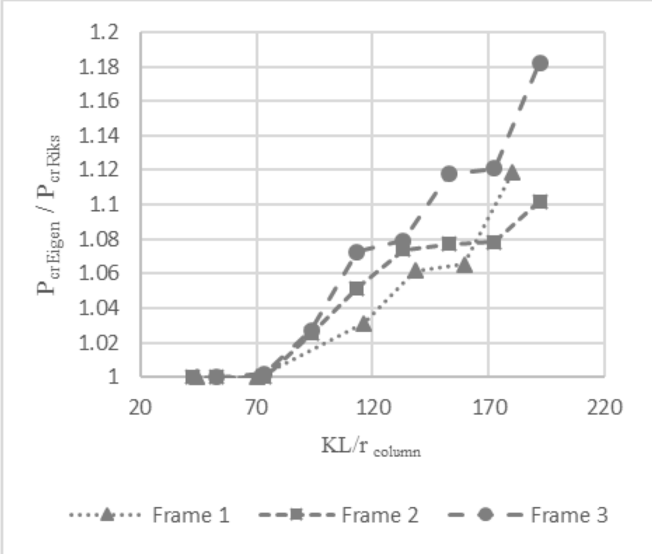

Figure 14: Ratio of $P_{cr\ Eigen} / P_{cr\ Riks}$ for different column slenderness ratio in sidesway uninhibited frame.

Table 1: Results of Sway frames loaded with line load

#	Case name	k	kl/r	Eigenvalue	P_{Riks}	Eigen/Riks
1	Frame 1	1.8989	44.47	2760.3	2760	1
2		2.0027	70.35	1736.6	1737	1
3		2.483	116.29	1214.6	1178	1.031
4		2.3606	138.20	902.41	850	1.061
5		2.2696	159.45	699.55	656.5	1.065
6		2.1996	180.29	559.54	500	1.119
1	Frame 2	2.4055	42.25	8521	8521	1
2		2.259	52.90	6116	6116	1
3		2.09	73.41	3457.9	3458	1
4		1.9958	93.48	2203.9	2150	1.025
5		1.9359	113.34	1525	1450	1.052
6		1.8942	133.08	1117	1040	1.074
7		1.8638	152.76	854.25	793	1.077
8		1.8408	172.43	674.08	625	1.079
9		1.8224	192.05	545.47	495	1.102

Table 2 continue:

#	Case name	k	kl/r	Eigenvalue	P_{Riks}	Eigen/Riks
1	Frame 3	2.2592	52.90	3126	3126	1
2		2.0903	73.43	2004	2000	1.002
3		1.9958	93.48	1387	1350	1.027
4		1.9359	113.34	1019	950	1.073
5		1.8942	133.08	782.48	725	1.079
6		1.8639	152.77	620.45	555	1.118
7		1.8409	172.44	504.51	450	1.1211
8		1.8226	192.07	418.56	354	1.182

Relation between critical loads obtained from linear analysis to corresponding value determined by the nonlinear Riks analysis are presented in Figure 15 for all sidesway uninhibited frames. Based on 2^{nd} order polynomial trendline, the results have been related as indicated in equation (5) below with a correlation coefficient, R^2, of 0.907,

$$P_{crEigen} = \left(5 \times 10^{-6}\left({kl}/{r}\right) - 2 \times 10^{-4}\left({kl}/{r}\right) + 0.9943\right)P_{crRiks} \qquad \ldots\ldots(5)$$

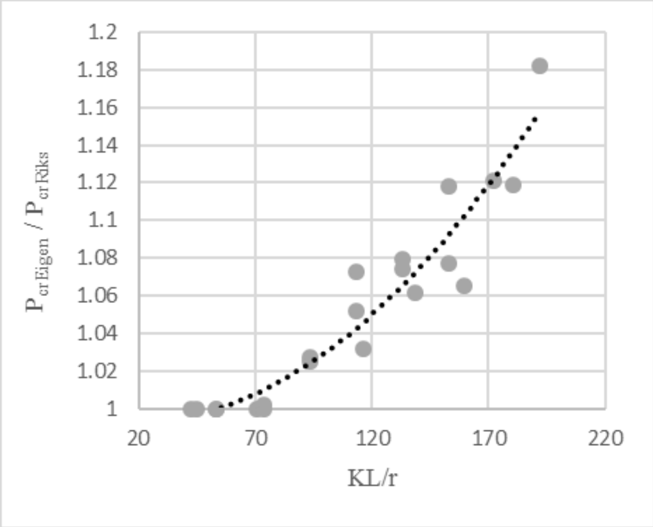

Figure 15: A scatter plot of $P_{cr\ Eigen}$ / $P_{cr\ Riks}$ ratio for different column slenderness ratio in sidesway uninhibited frame.

5.2. Sidesway inhibited Frame loaded with line load.

In this case, two types of frames has been simulated as follow;

- Frame 1: consist of HEB200 columns and IPE200 beam.
- Frame 2: consist of HEB200 columns and IPE270 beam.

The results indicated that; when the portal frame is sidesway inhibited, it will not be longer sensitive to imperfection, where the failure load specified by the nonlinear approach was caused by bending rather than buckling. As the slenderness ratio increased the frame bending stiffness decrease and that will cause the failure, therefore a large gap arise between the buckling load and failure load indicated by the Riks method as shown in Figure 16.

The failure mode shape and summary of the analysis results are presented in Figure 17 and Table 2 respectively.

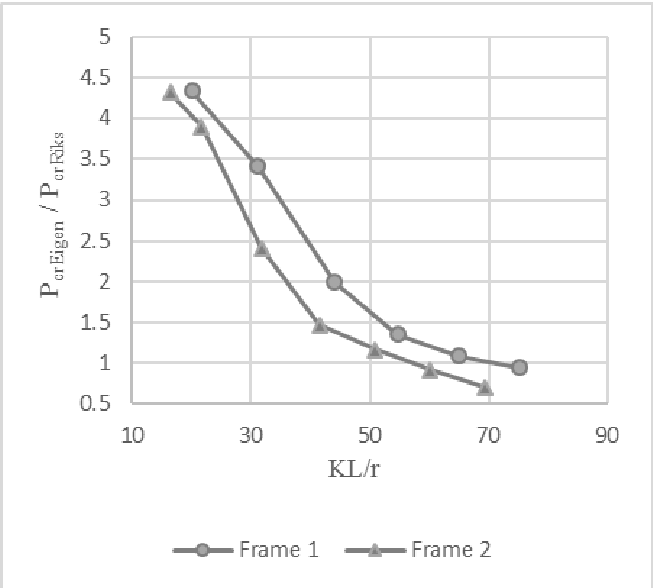

Figure 16: Ratio of $P_{cr\ Eigen}$ / $P_{cr\ Riks}$ for different column slenderness ratio in sidesway inhibited frame.

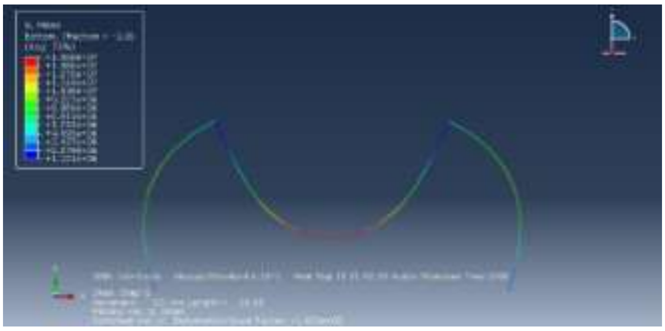

Figure 17: Failure mode indicated by the nonlinear Riks method.

Table 3: Results of non-sway frame loaded with line load.

#	Case name	k	kl/r	Eigenvalue	P_{Riks}	Eigen/Riks
1	Frame 1	0.858	20.10	15398	3550	4.337
2		0.88723	31.16	17601	5150	3.418
3		0.94227	44.13	12793	6411	1.995
4		0.93395	54.68	8883	6550	1.356
5		0.92611	65.06	6449.7	5920	1.089
6		0.91871	75.30	4887.6	5175	0.944
1	Frame 2	0.93725	16.46	23964	5550	4.318
2		0.92513	21.66	27699	7100	3.901
3		0.90386	31.75	22456	9300	2.415
4		0.88582	41.49	15021	10215	1.470
5		0.87035	50.95	10449	8950	1.167
6		0.85696	60.20	7657	8313	0.921
7		0.84527	69.28	5855.5	8330	0.703

5.3. Frame loaded with concentrated loads at columns top.

In this case, only one types of frames has been simulated, frame consist of HEB200 columns and IPE200 beam, due to noticing of a non-imperfection sensitive behavior as shown in Figure 18 and Figure 19 for sidesway inhibited and sidesway uninhibited respectively, where the critical load obtained by the linear Eigenvalue approach not exceeded 5% than the critical load obtained by nonlinear Riks method with a slenderness ratio, kl/r, reach up to 129, for sidesway inhibited frame, and 7% with a slenderness ratio, reach up to 223, for sidesway uninhibited frame. It should be noticed that the assigned geometric imperfection was not exceeded $L/1000$ as AISC recommendations for the maximum fabrication tolerance. Summary of analysis results are presented in Table 3.

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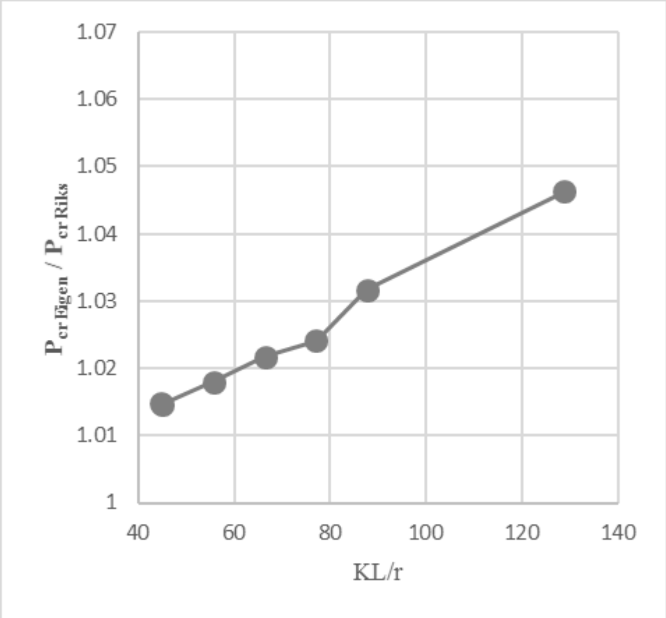

Figure 18: Ratio of $P_{cr\ Eigen}$ / $P_{cr\ Riks}$ for different column slenderness ratio in sidesway inhibited frame.

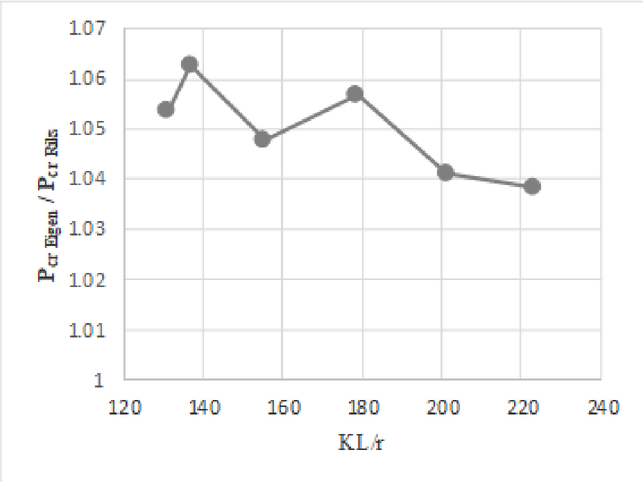

Figure 19: Ratio of $P_{cr\ Eigen}$ / $P_{cr\ Riks}$ for different columns slenderness ratio in sidesway uninhibited frame

Table 4: Results of sway and non-sway frame loaded with concentrated loads.

#	Case name	k	kl/r	Eigenvalue	P_{Riks}	Eigen/Riks
1	Sidesway inhibited	0.957	44.81	6979	6876.7	1.014876
2		0.961	44.99	6926	6825	1.014725
3		0.951	55.686	4553	4472.8	1.017953
4		0.946	66.446	3211	3143	1.021683
5		0.941	77.096	2391	2335	1.024111
6		0.936	87.642	1854	1796.6	1.031699
7		0.917	128.89	859.8	821.7	1.046306
1	Sidesway uninhibited	2.802	131.237	428.5	406.7	1.0536
2		2.924	136.952	371.95	350	1.0627
#	Case name	k	kl/r	Eigenvalue	P_{Riks}	Eigen/Riks
3	Sidesway uninhibited	2.657	155.573	324.83	310	1.0478
4		2.545	178.808	256.8	243	1.0568
5		2.455	201.232	209.27	201	1.0411
6		2.382	223.141	174.425	168	1.0382

6. Summary and Conclusions

This study pointed out how the analysis method could affect the buckling loads in sidesway inhibited and sidesway uninhibited steel portal frame, two analysis procedures with two load natures has been presented, linear eigenvalue buckling analysis and the nonlinear Riks analysis, where the maximum allowable members imperfection are included, has been performed. The frames were loaded with line load at its beams in some cases and loaded with concentrated loads at its columns in the other ones.

From different case studies the following conclusions have been drawn:

1. The critical loads obtained by the Eigenvalue buckling analysis could give an overestimate stiffness to the sidesway uninhibited steel portal frame loaded in its beam with uniformly distributed load, where the critical load obtained by the nonlinear Riks method could be less by 18% than the linear Eigenvalue buckling analysis.
2. When the portal frame is sidesway inhibited, and loaded in its beam with uniformly distributed load, it will not be longer sensitive to imperfection, where the failure load specified by the nonlinear approach was caused by bending rather than buckling. It has been noticed that as the slenderness ratio increased the frame bending stiffness decrease, which will cause the failure.
3. When the portal frame columns are loaded with concentrated loads, the frame didn't showed a sensitivity to geometric imperfection, where the ratio of critical load obtained by linear to nonlinear approaches give a range (1.48 – 4.6) % in the sidesway inhibited and (3.8 – 6.3)% for sidesway uninhabited.
4. An equation with correlation coefficient, R^2, equal to 0.907, that relate critical loads obtained from the linear eigenvalue buckling analysis with those obtained from the non-linear Riks method has been developed for portal frame loaded in its beam with uniformly distributed load.
5. For commercial software that offer only Eigenvalue buckling analysis to determinate buckling loads, the critical buckling loads obtained for steel portal frame can be reliable unless the frame was sidesway uninhibited and loaded with uniformly distributed load at its beam, as it showed sensitivity to geometric imperfection.

7. Recommendations

For future studies the following points are recommended

1. Investigate the imperfection sensitivity for portal frame where the bending is about column minor axes.
2. Investigate the imperfection sensitivity for other structural systems.

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