Paper 16

Proceedings of the Eleventh International Conference
on Computational Structures Technology,
B.H.V. Topping, (Editor),
Civil-Comp Press, Stirlingshire, Scotland

Imperfection Sensitivity Analysis of the Behaviour of Single Storey Frames with Tapered Web Members

I.M. Cristutiu[1] and D.L. Nunes[2]
[1]Faculty of Architecture
[2]Faculty of Civil Engineering
"Politehnica" University of Timisoara, Romania

Abstract

Ranging from short to long spans, the single storey steel frame, with pitched rafter, is one of the most commonly used types of structural system for building the main frame. As a result of both economic and aesthetical reasons, structural members (i.e. rafter and column) can be designed as welded "I" plate sections with tapered webs. The tapering is made according to the distribution of the bending moment along the major axis, resulting in a more economical use of the material. As a result of its non-prismatic shape, a slender web section is likely to occur at the maximum cross section height. Besides being subjected to important compressive axial forces, the length of the rafter is significant (depending on the span), hence the problem of stability is much more complex than in the case of multi-storey buildings. Sensitivity to the global frames imperfections, structural element imperfections and the possible coupling of these two is important when dealing with stability problems. If no lateral restraints, or when they are not effective enough, the lateral-torsional mode characterises the global behaviour of the frame members and interaction with sectional bucking modes may occur as well.
This paper summarises the outcome of a numerical study on a considerable number of such type of frames. A sophisticated nonlinear elastic-plastic finite element model and different types of initial imperfections were used to study the behaviour of the frames. Different types of lateral restraints (e.g. rigid and elastic) were also considered.

Keywords: manufacturing imperfections, assembling imperfections, pitched-roof portal frames, tapered members.

1 Introduction

Steel structural elements with variable cross section, made of welded plates, are largely used in construction industry for both beam and column in accordance with the stress and stiffness demand in the structure. Nonrectangular shape of the element might lead to thin/slender web section at its maximum height hence elastic to slender web results for the case of double T welded cross section. Due to their large relative slenderness about the minor axis, out of plane buckling usually governs their ultimate capacity. More than that, the out of plane buckling strength of these structures is directly influenced by the lateral restraining, end support and initial imperfections.

The design, execution and erection of steel structures must take place under certain limit constraints. If in the design process, one must ensure strength, stability and rigidity to the structure, in the manufacturing and erection process certain admissible tolerance limits must be accounted for. EN 1090-2[2] is the European standard that establishes the values of admissible limit tolerances for the manufacturing and erection of steel structures. The last version allows for a maximum linearity deviation of manufactured/erected elements a value of L/750 (L=length of the element). This is less strict than previous requirements (L/1000), which were the basis for the present European buckling curves (EN 1993-1 [1]). The European norm contains provisions for several initial imperfections (vertical deviation and initial arc imperfections), which take values function of the considered buckling curve (a0, a, b, c, d) and buckling mode (minimum or maximum axis). If the values for manufacturing/erection allowable tolerances are higher than those of the above mentioned imperfections, further investigations are required in order to establish if the partial safety factor (γ_{M1} = 1,00), given in SR-EN 1993-1-1 is still enough.

Steel structural elements with variable cross section, made of welded plates, are largely used in construction industry for both beams and columns in accordance with the stress and stiffness demand in the structure. Due to nonrectangular shape of the element, thin web section may be obtained at the maximum cross section height. Usually class 3 to class 4 web results for the case of double T welded cross section. The buckling strength of these structures is directly influenced by the lateral restraining, end support and initial imperfections [1]. If no lateral restrains, or when they are not effective enough, global behaviour of frame members is characterized by the lateral torsional mode and interaction with sectional buckling modes may occur.

The main objective of the paper is to analyse the sensitivity of single storey steel structures made of variable cross section to different type of lateral restraints and supplementary to manufacturing and erection imperfections. The previous studies [3,4,5,6,7] made by several authors all around the world, highlighted the importance of taking into account different initial imperfections, even in case of gravitational loads and horizontal loads. The considered imperfections might be described as: column vertical deviation (in or out-of-plan), initial bow imperfections, cross sectional imperfections, coupling between previously defined imperfections. Finally the following problem arises: what is more important when dealing with stability problems the type or value of imperfection?

2 Imperfection and tolerances

Whether there are imperfections generated by the fabrication process or the assembling process, we can say for certain that "All structures are imperfect". The structural imperfections can be divided in two large categories, imperfections generated by the fabrication process and imperfections generated by the assembling process namely the global imperfections.

According to [1,2] appropriate allowances should be incorporated in the structural analysis to cover the effects of imperfections, including residual stresses and geometrical imperfections such as lack of verticality, lack of straightness, lack of flatness, lack of fit and any minor eccentricities present in joints of the unloaded structure. The assumed shape of global imperfections and local imperfections may be derived from the elastic buckling mode of a structure in the plane of buckling considered. Both in and out of plane buckling including torsional buckling with symmetric and asymmetric buckling shapes should be taken into account in the most unfavourable direction and form.

The structural imperfections can be divided in two large categories, imperfections generated by the fabrication process, which include the material imperfections, the geometrical imperfections at the level of the structural elements and subassemblies and imperfections generated by the assembling process namely the global imperfections.

In Figure 1 are presented two types of imperfections recorded on site: a) erection imperfections (out of plane rafter displacement); b) manufacturing imperfection (local buckling of the web).

a) b)

Figure 1. Imperfections recorded on site: a) erection imperfection (er); b) manufacturing imperfection (man).

For reducing to a minimum the structural imperfections in the fabrication and assembling process, their level is limited by quality standards and norms and in design the effect of imperfections is considered through safety coefficients and special design procedures. For frames sensitive to buckling in a sway mode the effect of imperfections should be allowed for in frame analysis by means of an equivalent imperfection in the form of an initial sway imperfection and individual bow imperfections of members [1,2]

3 Analysed frames

A number of pitched roof portal frames commonly used for the execution of single story buildings were analysed. They are of different spans and heights (see Figure 1), to cover a wide range. The frames were designed to verify the ULS and SLS criteria under the gravitational loads. They have pinned column base, tapered columns, tapered rafters and a pitch roof angle of 8^0. Tapering ratio, h_{max}/h_{min}, is approximately equal to 2 for all the cases. The rafter is composed by both uniform and non-uniform regions, as can be seen from Figure 1. The length of the rafter haunch is 15% from the span in all the cases. The main dimensions of characteristic sections of frames are presented in Table 1. The chosen dimensions are quite common in practical applications.

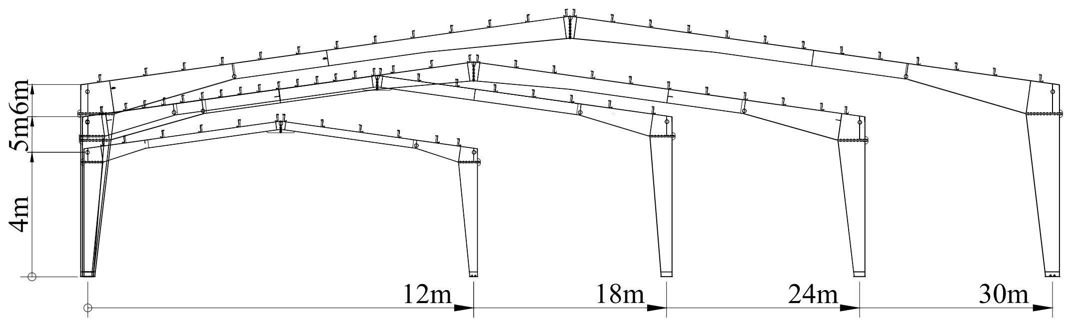


Figure 2. Geometry of the analysed frames

Code	*H [m]*	*L [m]*	*Dimensions $h*b*t_f*t_w$ [mm]* tapered column	*tapered rafter*	*rafter*
4x12	4	12	(250…600)*200*10*8	(260…500)*150*10*8	260*150*8*6
5x12	5	12	(250…600)*220*10*8	(260…500)*150*10*8	260*150*8*6
6x12	6	12	(250…600)*240*10*8	(260…500)*150*10*8	260*150*8*6
4x18	4	18	(350…700)*250*12*10	(360…700)*200*12*10	360*200*10*8
5x18	5	18	(350…700)*250*14*10	(360…700)*200*12*10	360*200*10*8
6x18	6	18	(350…700)*260*14*10	(360…700)*200*12*10	360*200*10*8
4x24	4	24	(350…850)*270*14*10	(440…850)*240*14*10	(440...600)*240*12*8
5x24	5	24	(350…850)*270*14*10	(440…850)*240*14*10	(440...600)*240*12*8
6x24	6	24	(350…850)*300*14*10	(440…850)*240*14*10	(440...600)*240*12*8
4x30	4	30	(450…1050)*310*14*12	(500…1050)*270*16*12	(500...700)*270*12*8
5x30	5	30	(450…1050)*310*14*12	(500…1050)*270*16*12	(500...700)*270*12*8
6x30	6	30	(450…1050)*340*14*12	(500…1050)*270*16*12	(500...700)*270*12*8

Table 1. Main dimensions of the analysed frame

Both eigen-buckling (LEA) and nonlinear elastic-plastic (GMNIA) analyses had been applied according to EN1993-5 [8] and EN1993-1. The computation was performed with Abaqus FEM [9] program using Shell elements enabling for large plastic deformation. Joints were modelled using contact area-to-area elements in order to simulate the real behaviour of the connections in the global analysis (see Figure 3). The material behaviour was introduced by a bilinear elastic-perfectly plastic model, with S355 yield strength (see Figure 3). Lateral restrains by purlins were also considered (see Figure 4). Moreover, in all these cases, the restraining effect induced by longitudinal beams located at eaves and ridges were modelled.

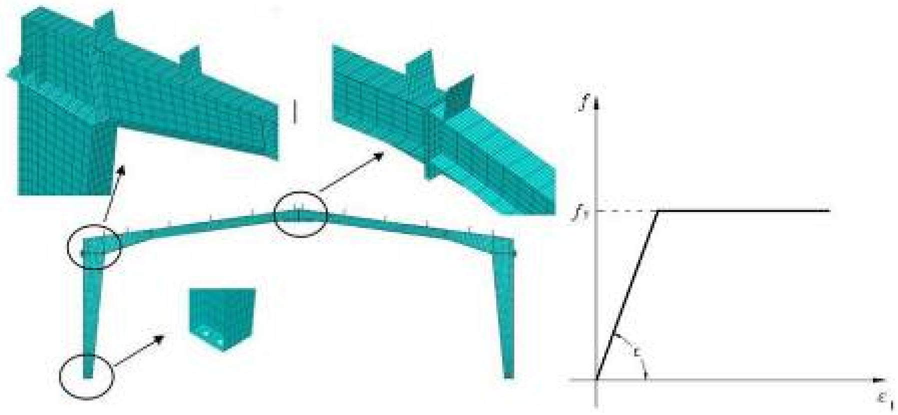
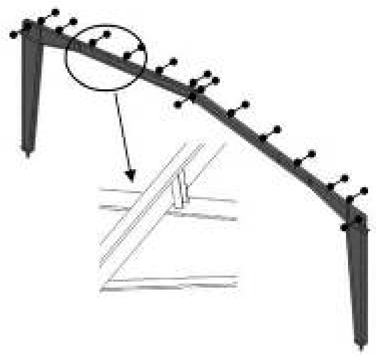
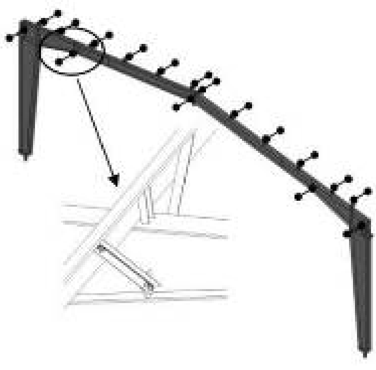


Figure 3. FEM modelling of the analysed frames and material behaviour

a) restraint type 1 (res-1)



b) restraint type 2 (res-2)

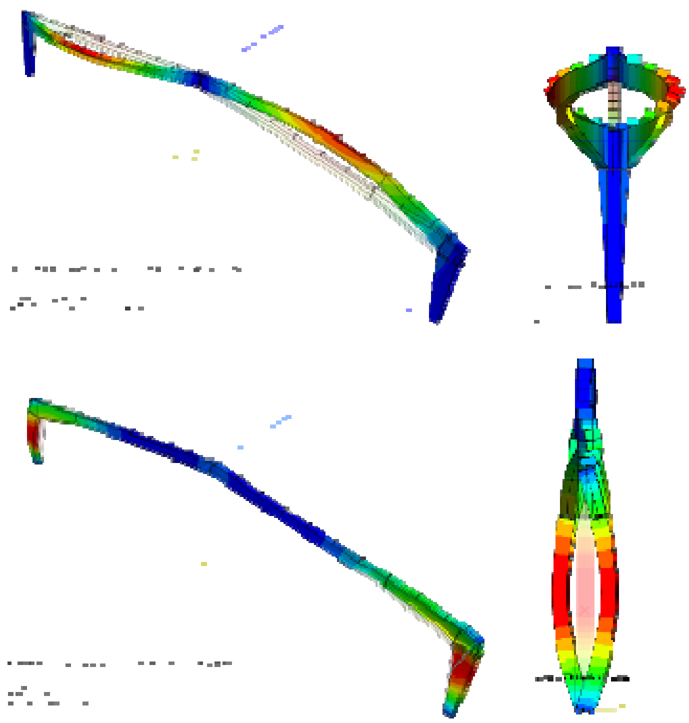


c) restraint type 3 (res-3)

Figure 4. Type of lateral restraints and their location

The lateral restraints are of 3 different types, as shown in Figure, and were applied in two different ways.- “rigid and elastic”. Type 2 represents the purlin/sheeting effect, when the purlin is pinned when intersecting the rafter. Type 3 is similar with type 2 with an additional fly brace. Type 1, the reference case, actually means no lateral restrains introduced by purlins. At first, to simplify the computational model, in the analysis the lateral restrains had been considered axially rigid. The actual behaviour of the purlins (Z150/1.5) was considered later on to identify the difference between the rigid and elastic cases.

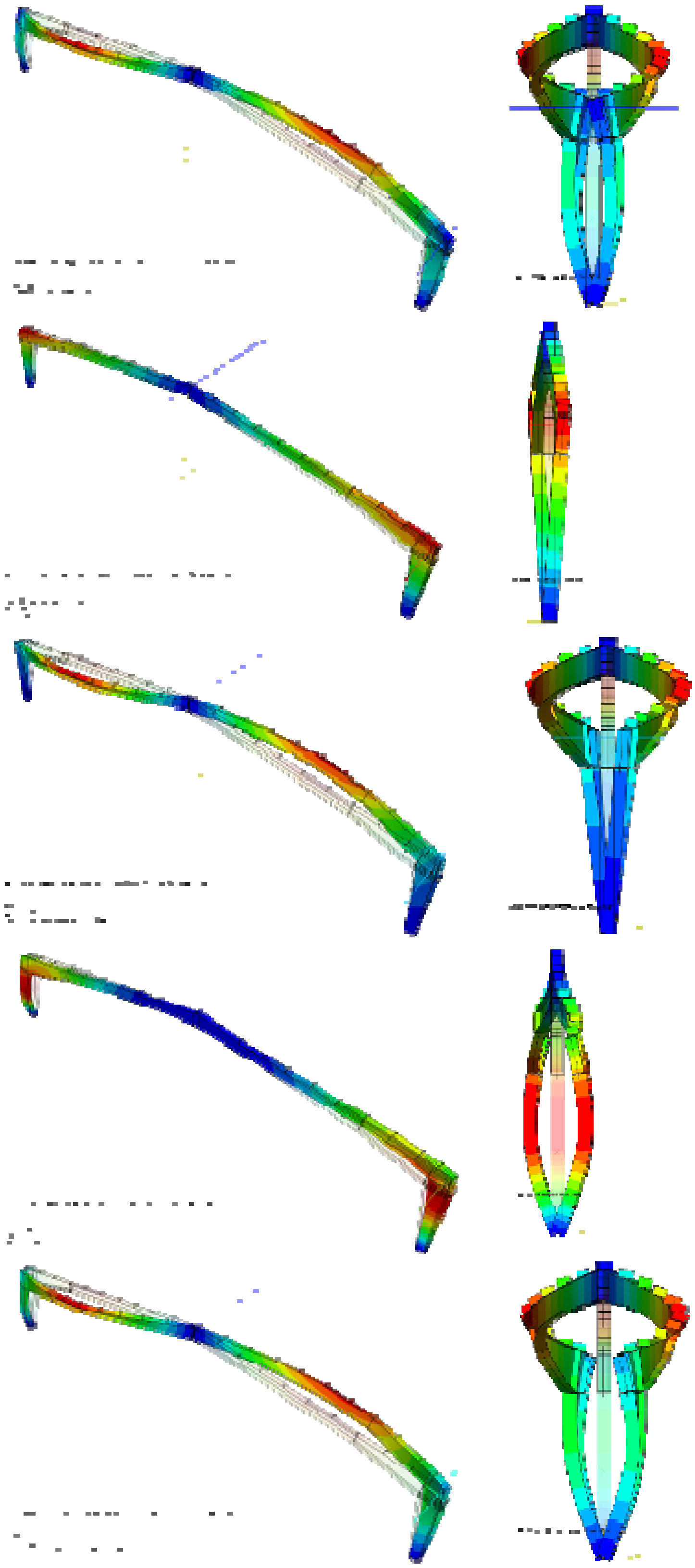
Rafter-to-column and rafter-to-rafter connections are bolted with extended and plates toward exterior as shown in Figure 2. Vertical loads from permanent and snow actions were introduced at the purlin location (e.g 1.2 m along the rafter). The applied imperfection are presented in Figure 5, and is characterised by an initial out of plane bow displacement for the case of manufacturing imperfections and top column out of plane displacement for the case of assembling imperfection. This type of imperfection is slightly different from those applied on bar elements, where perfect bending might be applied. Herein twisting of the element was also recorded, that represents the real case. The amplitude of the considered manufacturing, equal with l/150 (where l represents the length of the element), are presented in Figure 5 as well.



a)
Manufacturing imperfection – initial bow of the rafter
-40 mm (12 m span)
-60 mm (18 m span)
- 80 mm (24 m span)
- 100 mm (30 m span)

b)
Manufacturing imperfection – initial bow of the column
-26.67 mm (4 m height)
-33.33 mm (5 m height)
-40.00 mm (6 m height)

Figure 5: Manufacturing and assembling imperfections considered in the analyses



c)
Manufacturing imperfection – initial bow of the rafter and column (a+b)

d)
Assembling imperfection initial sway displacement
-14 mm (4 m height),
-17.5 mm (5 m height)
-21 mm (6 m height)

e)
Manufacturing imperfection – initial bow of the rafter + assembling imperfections initial column sway (a+d)

f)
Manufacturing imperfection – initial bow of the column + assembling imperfections initial column sway (b+d)

f)
Manufacturing imperfection – initial bow of the rafter and column + assembling imperfections initial column sway (a+b+d)

Figure 5: (cont.) Manufacturing and assembling imperfections considered in the analyses

4 Results of numerical analyses

4.1 Case of elastic/actual restraints

3D GMNIA and 3D LEA analysis were performed to identify the failure of the frames and their elastic buckling behaviour. For the GMNIA analysis, initial of plane imperfections as the ones presented in Figure 5 were considered. The critical load multipliers and ultimate load multipliers corresponding to the eigen-buckling shape and failure of the structure respectively were determined for all analysed frames. In the analysis lateral restraints were taken into account, as actual elastic ones. Herein, due to lack of space only the case of Hx12pin and Hx18pin are presented in Figure 6.

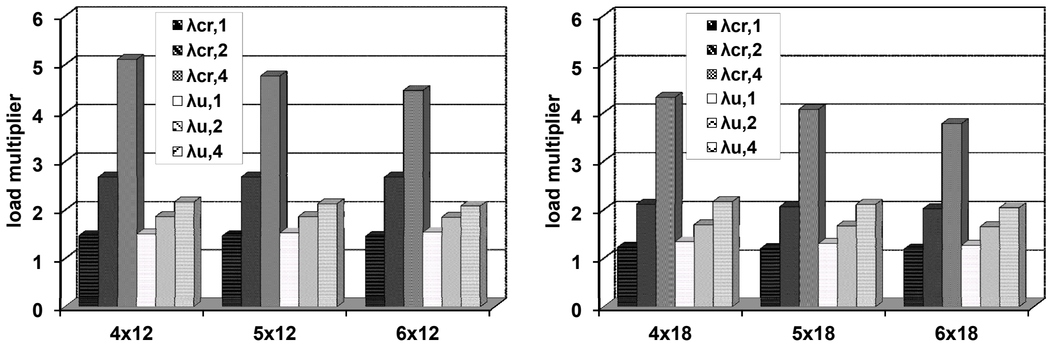
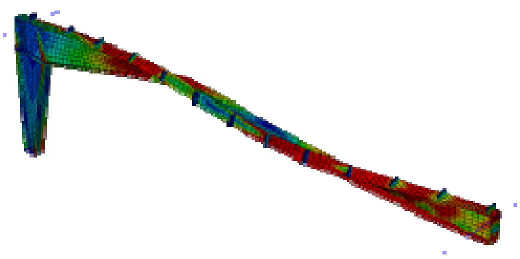
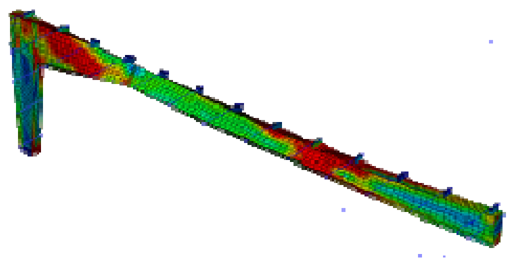


Figure 6. Load multipliers for LEA and GMNIA analyses.

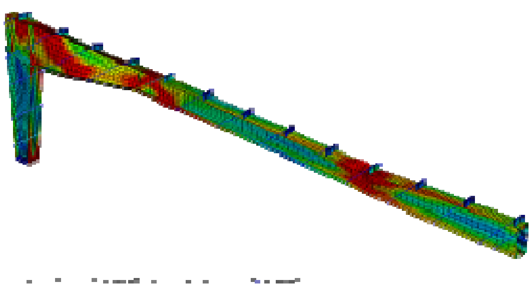
In Figure 7, are illustrated the failure modes corresponding for GMNIA analysis for the 3 type of lateral restraints.



a) restrain type 1



b) restrain type 2



c) restrain type 3

Figure 7. Plastic sectional buckling - GMNIA analysis (Von Misses stress distribution scale factor 1)

Analyzing these results one observes that lateral restraints influence the buckling shape and failure of the structural elements. In several cases, when the structure is well laterally restrained (case 3 restraints of Figure 4), local buckling of the web may develop prior to lateral-torsional mode. The critical local buckling load in cases of restrains type 3, is higher than the ultimate load obtained from elastic-plastic analysis, λ_u (see Figure 6). We can say in this case that the structure may fail due to local plastic sectional buckling instead of elastic overall buckling.

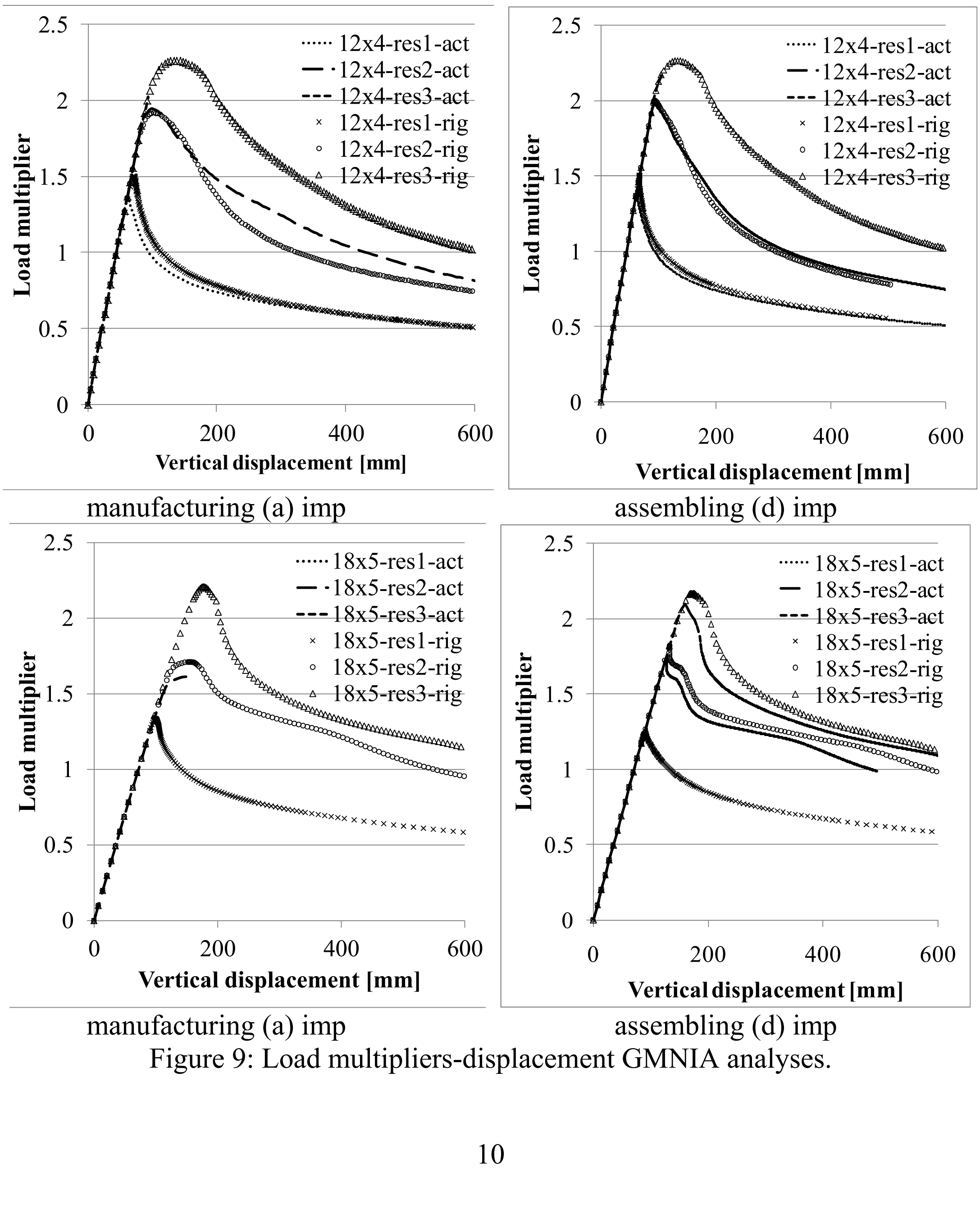
In Figure 8 a comparison of the ultimate load multiplier, of the GMNIA analysis, is presented distinct for different type of frame configuration and initial imperfections: 12x5, 18x4, 24x5, 30x6.

Figure 8. Ultimate load multiplier - GMNIA analysis – accounting for different type of initial imperfections

From Figure 8, it can be concluded that the ultimate capacity of the frame is directly influence by the type of lateral restraints and initial imperfection. The imperfections that influences the most the bearing capacity of the frames are those obtained by combination of simple ones (e.g. c- Manufacturing imperfection – initial bow of the rafter and column a+b and g - Manufacturing imperfection – initial bow of the rafter and column + assembling imperfections initial column sway a+b+d).

4.2 Case of rigid lateral restraints

In Figure 9, a comparison between perfect structure, initial bow (out of plane) imperfection (a) and initial sway imperfection (d) for elastic and rigid restraint is presented.



manufacturing (a) imp assembling (d) imp

manufacturing (a) imp assembling (d) imp

Figure 9: Load multipliers-displacement GMNIA analyses.

The two types of imperfections (a and d from Figure 5) considered in analysis influence the bearing capacity of the considered structures, depending on the applied lateral restraints. The considered imperfections do not influence the initial rigidity of the structure. The influence is much higher in case of low restrained structure (type 1 and 2 in Figure 4) than in the case of well restrained ones (type 3 in Figure 4). In the case of lateral-torsional buckling, which represents the natural coupling between flexural and torsional modes, the actual buckling strength is characterized by a low-to-significant erosion of theoretical one, function of the lateral restraints. The difference between the rigid restraints and elastic ones is quite small, 0-10%, for all analysed cases with a slight increase from small to large span.

5 Conclusion

A parametric study was made in order to analyse the sensitivity of the out-of- plane imperfections in the behaviour of pitched-roof portal frames made of elements with tapered web. For this purpose seven different types of imperfections (e.g. manufacturing imperfections characterized by initial bow imperfections, and erection imperfections characterized by initial sway imperfections) and different type of lateral restraints (rigid and actual) were considered. The magnitude of the imperfections was set equal with those prescribed in EN1993-1-1 [1].

For all the cases out-of-plane buckling of the frame elements was noticed to be the main failure mode indifferent of the applied lateral restraints. There were cases for which the global lateral- torsional buckling of the frames was coupled with local buckling of the web. This was mainly observed when the restraints applied on the frame element are more effective against overall buckling (e.g. type 3 restraints). It was noticed that the considered imperfections has a low to significant influence on the final capacity of the frame, function of the applied lateral restraints. The difference between the imperfections considered is significant mainly for the combined cases.

The difference between elastic (actual) and rigid lateral restraints increases by the span increasing, a maximum 10 % difference was recorded.

Acknowledgement

The financial support of “National University Research Council – NURC- Romania” through the national research grant PN-II-RU-TE-2010-1/38 is gratefully acknowledged.

References

[1] EN 1993-1-1, “Eurocode 3 —Design of steel structures Part 1.1: General rules and rules for buildings”, CEN - CEN - Brussels, Belgium 2005.

[2] EN 1090-2:2005 Execution of steel structures and aluminium structures - Part 2:Technical requirements for the execution of steel structures. CEN-Brussels

[3] Z. Kala (2011), “Sensitivity Analysis of Steel Plane Frames with Initial Imperfections”, *In Engineering Structures*, Vol.33, No.8, 2011, pp. 2342–2349.

[4] Z.Kala (2011) “Sensitivity Analysis of Stability Problems of Steel Plane Frames”, *In Thin–Walled Structures*, Vol.49, No.5, 2011, pp. 645–651.

[5] A. Taras, and R Greiner, “Torsional and flexural torsional buckling — A study on laterally restrained I-sections”, Journal of constructional steel research, 64 (2008) 7-8, S. 725 – 731, 2008.

[6] J.Szalai and F.Papp, “On the probabilistic evaluation of the stability resistance of steel columns and beams”, Journal of Constructional Steel Research, 65 (2009), 569-577,2009.

[7] B. Farshi and F. Kooshesh, “Buckling Analysis of structural steel frames with inelastic effects according to codes”, Journal of constructional steel research, 65 (2009), 2078-2085, 2009.

[8] EN 1993-1-5, “Eurocode 3 (2003) —Design of steel structures Part 1.5: Plated structural elements”, CEN - CEN - Brussels, Belgium 2003.

[9] EN 1090-2:2005 Execution of steel structures and aluminium structures - Part 2:Technical requirements for the execution of steel structures. CEN-Brussels