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Influence of lateral restraints modeling in pitched-roof single storey frames with tapered members considering initial imperfections

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Abstract

The portal frame is one of the most commonly used type of structural system for span ranging up to 40m and has been used for frames up to 75m span. In practice it is found that the most efficient solution is generally obtained by using class 2 or 3 flanges and class 3 or 4 webs. Whilst its economic advantages, this refinement brings additional designing challenges: the high slenderness of its resulting cross-sections together with the uncertainty of results caused by the increased difficulty of a thorough calculation of the resistance to lateral-torsional buckling of tapered I-section elements, increases this structure's proneness to stability issues. When class 4 sections are used, which generally is the case of the rafter in the hunched part, the sectional buckling (e.g. local buckling of walls or distortion) may occur in elastic domain. If no lateral restraints, or when they are not effective enough, the lateral torsional mode characterizes the global behavior of frame members and, again, interaction with sectional bucking modes may occur. Moreover, as higher classes are more sensible to buckling, they are more sensitive to imperfections than more compact sections are. The actual design codes do not cover the practical design of this aforementioned type of structure. There are some provisions for design, but they are either too pessimistic or not cover all practical applications. This paper summarizes a numerical study performed by authors on a relevant series of such type of elements. A sophisticated nonlinear inelastic FEM model and different imperfections have been used to simulate the behavior of the elements.

Rezumat

Cadrul tip portal este unul dintre cele mai folosite sisteme structurale pentru a acoperi deschideri pana la 40 m. Practic, solutia cea mai eficienta se obtine folosind talpi de clasa 2 sau 3 si inimi de clasa 3 sau 4. In ciuda avantajelor economice, acest rafinament aduce provocari aditionale in proiectare: zvelteta inalta a sectiunilor rezultante impreuna cu incertitudinea datorita dificultatii sporite a unui calcul rigoros al rezistentei flambajului prin incovoiere-rasucire in elemente tip I cu sectiune variabila accentueaza riscul de probleme de stabilitate. Atunci cand se folosesc sectiuni de clasa 4, situatie intalnita des la vuta, flambajele distortional si local pot avea loc in domeniul elastic. In cazul in care nu sunt prevazute legaturi laterale sau atunci cand eficienta lor nu este asigurata, modul de flambaj prin incovoiere-rasucire caracterizeaza comportarea globala a cadrului si se poate dezvolta si o interactiune cu modurile sectionale de flambaj. Pe deasupra, cu cresterea clasei creste si sensibilitatea la flambaj si, in consecinta, sensibilitatea la imperfectiuni.
Normativele de proiectare actuale nu cuprind o proiectare practica pentru o astfel de structura. Exista unele prevederi dar sunt sau prea conservative sau nu cuprind toate aplicatiile practice.
Acest articol descrie studiul numeric facut de catre autori pe astfel de elemente. Pentru a simula comportarea elementelor s-a folosit un model neliniar inelastic pe baza MEF si diferite tipuri de imperfectiuni.

Keywords: Stability, buckling, lateral restraint, tapered element, imperfections, single-storey

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 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). 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. According to [1,2] appropriate allowances should be incorporated in the structural analysis to cover the effects of imperfections. 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.

a)

b)

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

Both in and out of plane buckling including torsional buckling with symmetric and asymmetric buckling shapes should be taken into account in the most unfavorable 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).
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].

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 and coupling between previously defined imperfections.

2. Analysed frames

For the purpose of this study a number of pitched roof portal frames of different spans and heights (see Figure 2) were analysed. 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 °. Tapering ratio, h_{max}/h_{min}, is approximately equal to 2 for all the cases. The rafter is composes by both uniform an non-uniform regions, as can be seen from Fig. 2. 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.

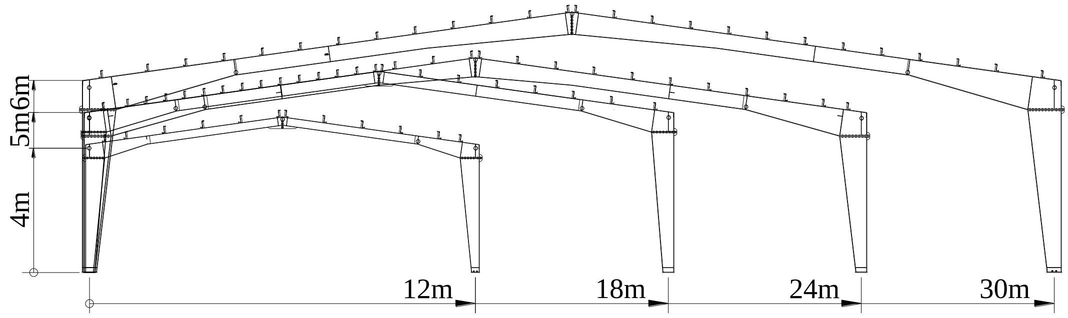


Figure 2. Geometry of the analysed frames

For the analysis of the objects in study was developed a finite element model capable of both geometrical and material nonlinearities. Both linear Eigen buckling (LEA) and nonlinear elastic-plastic considering geometric nonlinearities (GMNA and GMNIA) analyses [5] have been applied using the finite element program ABAQUS. S4R-type elements were used (4-node shell elements with reduced integration) with 6 degrees of freedom on each node (translation and rotation in regard to the x, y and z-axis).

Table 1: Main dimensions of the analyzed frame

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

To determine the structural response of the nonlinear problem an implicit solution strategy was used. A load stepping routine was used in which the increment size follows from accuracy and convergence criteria. Within each increment, the equilibrium equations are solved by means of the Newton-Raphson iteration (through the arc-length method).

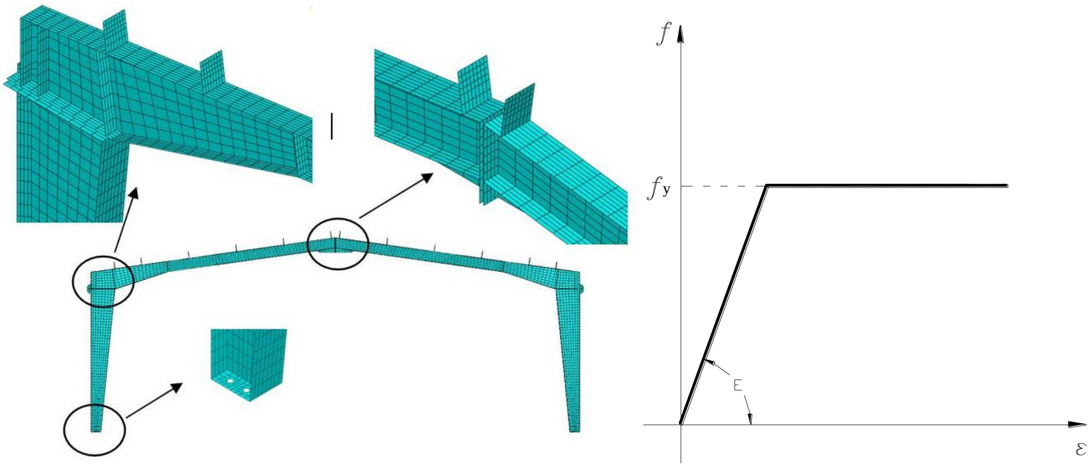
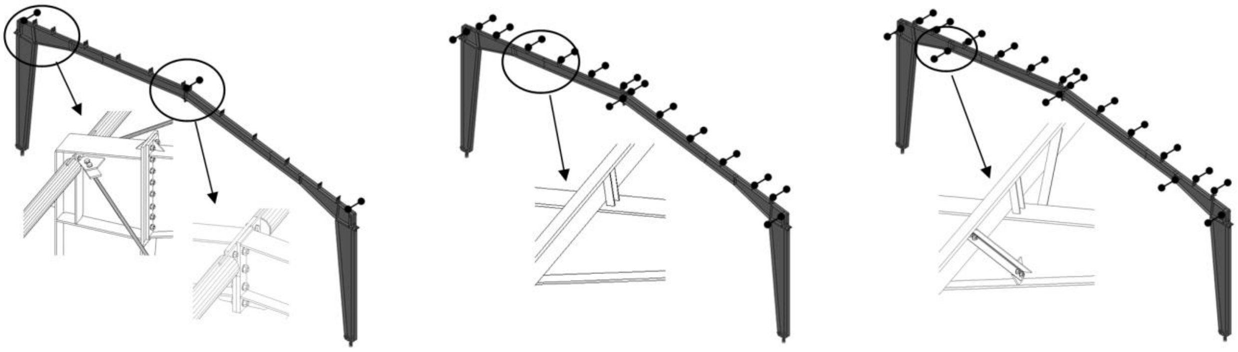


Figure 3. FEM modeling of the analyzed frames and respective elastic-perfectly plastic material behaviour

All plates were modeled in their mid-plane and the connections between beams and between beams and columns were defined as a surface-to-surface tie between both end-plates (Figure 3). The model's material was defined as elastic – linear plastic (E = 210000 N/mm2, υ = 0,3, fy = 355 N/mm2) and vertical loads from permanent and snow actions were introduced at the purlin location (e.g 1.2 m along the rafter). In all these cases, it was simulated the restraining effect induced by longitudinal beams located at eaves and ridges.

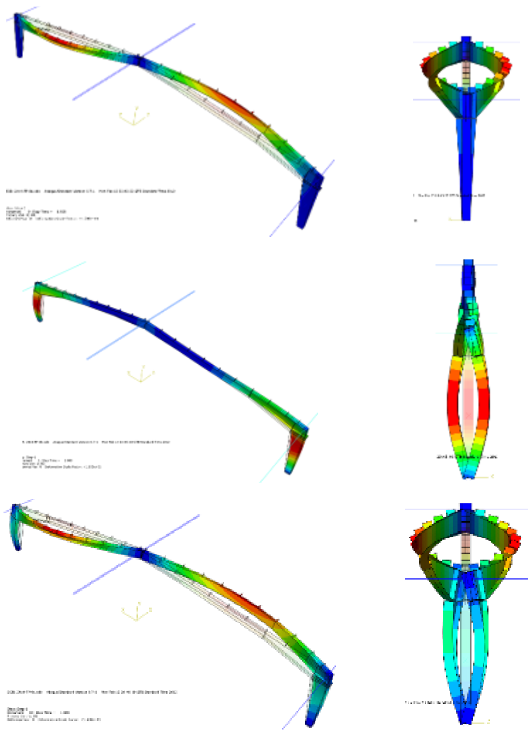


a) Restraint type 1 (res-1) b) Restraint type 2 (res-2) c) restraint type 3 (res-3)

Figure 4. Type of lateral restraints and their location

The lateral restraints applied are of 3 different types, as shown in Figure 4, and were applied in two different ways rigid and elastic. Types 2 represent 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, 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).

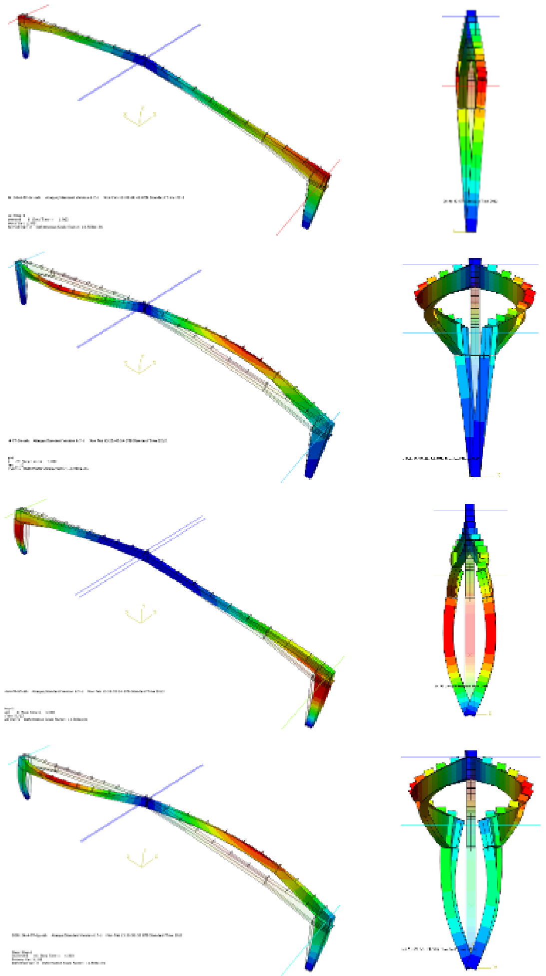
A set of both erection and manufacturing imperfections were considered separately in analyses. The applied imperfections are presented in Figure 5. Using shell elements, the imperfections are slightly different from those applied on bar elements, where perfect bending or perfect inclination might be applied. Herein twisting of the element was also recorded, that represents the real shape of imperfect elements. 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)

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. Manufacturing and assembling imperfections considered in the analyses

3. Results of numerical analyses

In order to identify the failure of the frames and their elastic buckling behaviour 3D GMNIA and 3D LEA analysis were performed. 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.

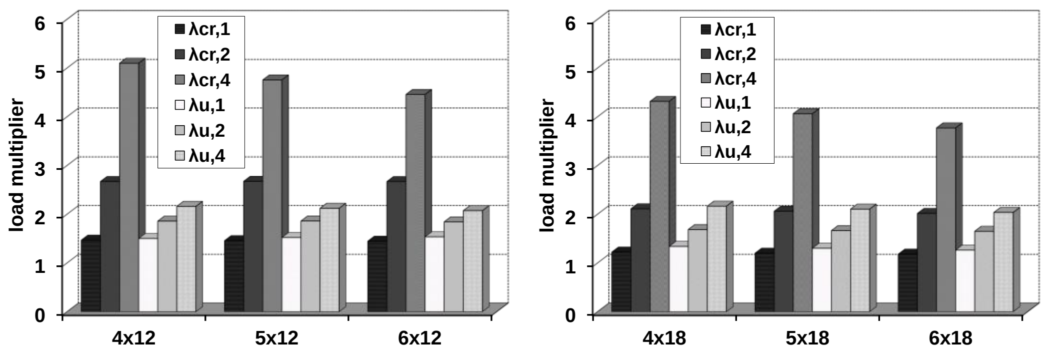
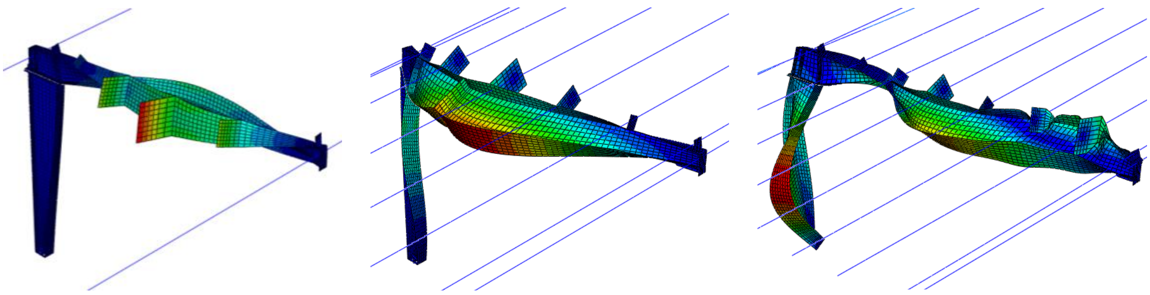


Figure 6. Load multipliers for LEA and GMNIA analyses

In Figure 8 7, are illustrated the failure modes corresponding for LEA analysis for the 3 types of lateral restraints



a) restrain type 1 b) restrain type 2 c) restrain type 3

Figure 7. First buckling shapes of LEA analysis.

and in Figure 8, are illustrated the failure modes corresponding for GMNIA analysis for the same 3 types of lateral restraints.

a) restrain type 1

b) restrain type 2

c) restrain type 3

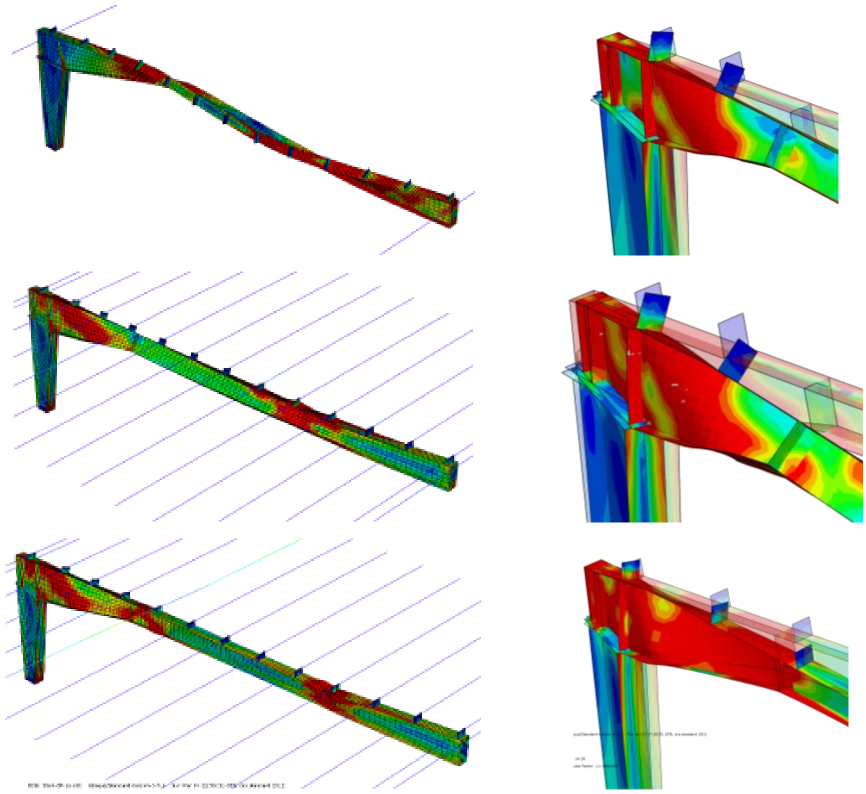


Figure 8. Failure modes - GMNIA analysis (Von Misses stress distribution scale factor 1)

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 the 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.

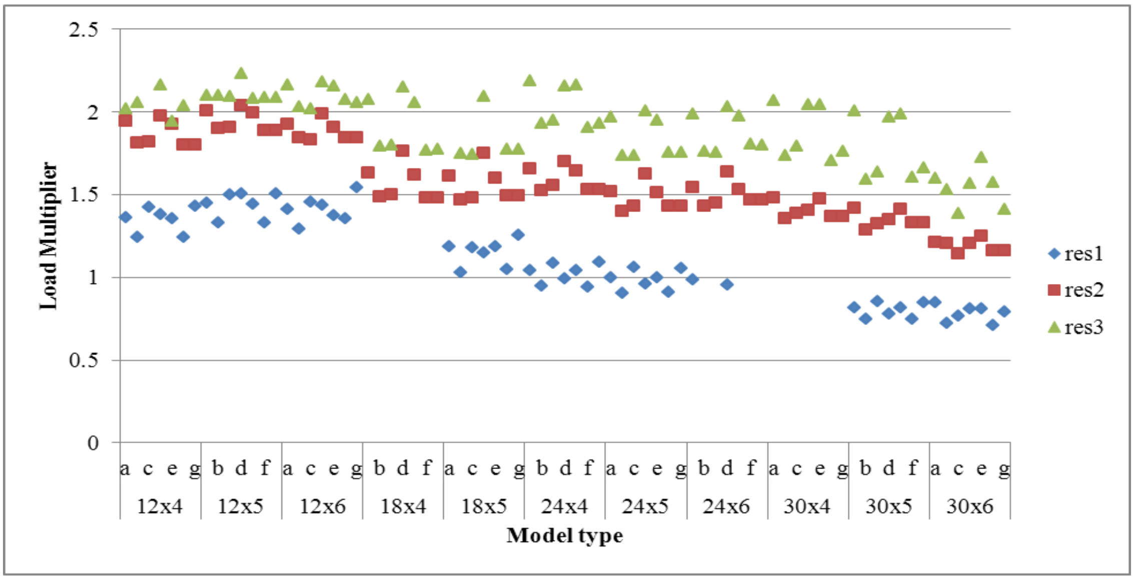
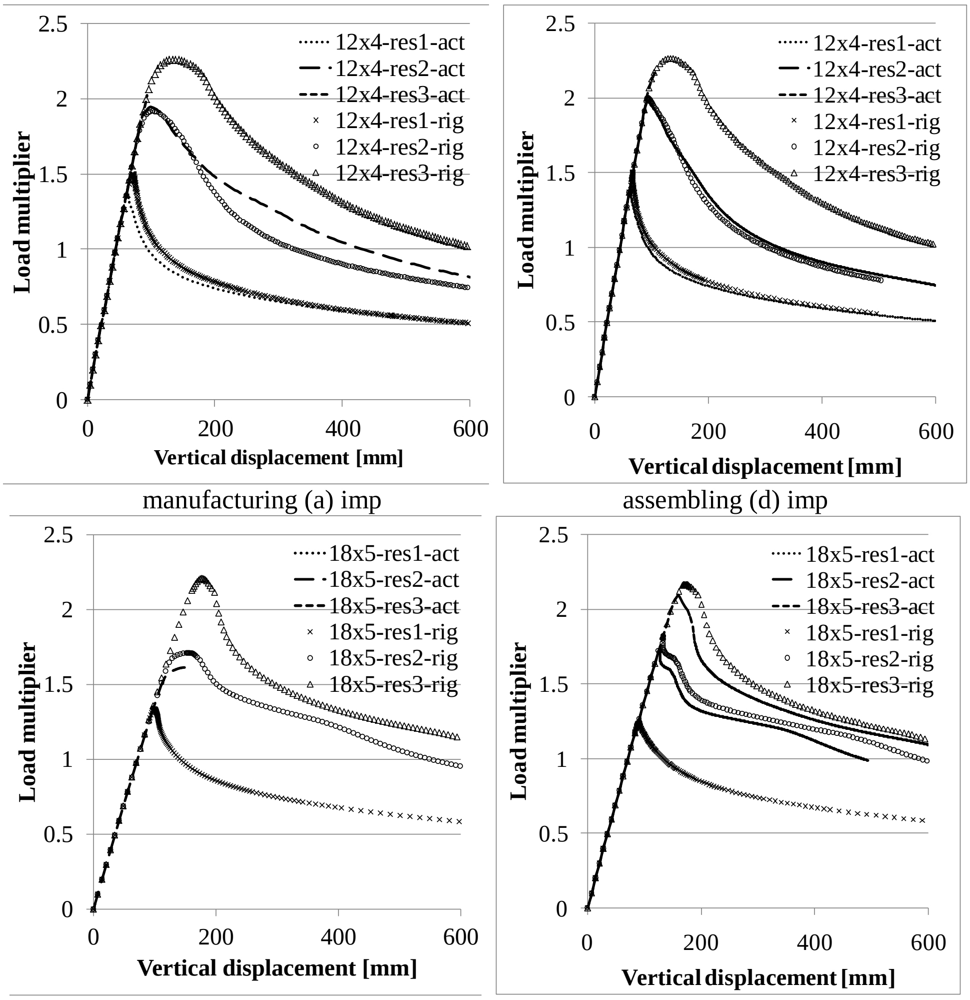


Figure 9. Ultimate load multiplier - GMNIA analysis – accounting for different type of initial imperfections



manufacturing (a) imp | assembling (d) imp

manufacturing (a) imp | assembling (d) imp

Figure 10. Load multipliers-displacement GMNIA analyses

In Figure 9 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.

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).

3.1 Comparison with rigid lateral restraints

In Figure 10, a comparison between elastic and rigid restraint is presented for initial sway imperfection (d). The imperfection (d from Figure 5) considered in analysis influence the bearing capacity of the considered structures, depending on the applied lateral restraints although they do not influence their initial rigidity. 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). 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.

4. Conclusion

A parametric study was made in order to analyse the sensitivity of different types of lateral restraints on pitched-roof portal frames made of elements with tapered web with out-of-plane initial imperfections. For this purpose seven different types of imperfections and three different types of lateral restraints 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 observed to be the main failure mode indifferent of the applied lateral restraints, although there were cases in 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 considered imperfections 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.

Acknowledgements
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