Behaviour of single storey frames with tapered web members accounting for manufacturing and assembling imperfections

I. MIRCEA CRISTUTIU
Department of Architecture
“Politehnica” University of Timisoara
Timisoara-300223, Traian Lalescu str., No. 2/602
ROMANIA
mircea.cristutiu@arh.upt.ro

DANIEL. L. NUNES
Department of Steel Structure and Structural Mechanics
“Politehnica” University of Timisoara
Timisoara-300224, Ioan Curea str., No. 1
ROMANIA
dln.nunes@gmail.com

Abstract: - Modern industrial halls and not only, posses a single storey steel frame usually made of welded plate elements with tapered web classified as class 3 and 4. In case of class 3 sections, when they are restrained against lateral or/and torsional buckling, the interaction between sectional plastic buckling and overall elastic buckling of the members in compression and/or in bending is possible. When class 4 sections are used, which generally is the case of the rafter in the maximum height of the tapered web, 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 enough effective, the lateral torsional mode characterizes the global behavior of frame members and, again, interaction with sectional buckling modes may occur. The paper summarizes a numerical study performed by authors on a relevant series of such type of frames. A series of 6 frames of different spans and heights have been analyzed. The frames were designed to withstand the vertical loads and satisfy the ULS and SLS criteria. Also different types of lateral restraints had been considered within the analysis. For the purpose of the work, nonlinear elastic plastic analyses and eigen buckling analyses with FEM were performed.

Key-Words: - assembling and manufacturing imperfections, steel pitched roof portal frames, tapered members, stability, lateral restraints.

1 Introduction
Pitched roof portal frames usually used in construction industry for industrial steel buildings are currently fabricated by slender welded sections of class 3 and/or 4. Frame members are of variable cross-section, in accordance with stress and stiffness demand.

Because rafters carry significant axial compressive loads, the problem of stability is more complex than in case of multi-storey frames [1]. If no adequate restraints are provided, the lateral torsional buckling strength of the members is generally low. Purlins and side rails supporting the roof deck and cladding introduce some restraining effect, but it is difficult to quantify it for usual design.

Actual design codes do not cover the practical design of this kind of structure. There are some provisions for design [2][3], but they are either too pessimistic or not cover all practical applications. Even so, the existing design methods are oriented on isolated elements only. In case of this type of structures the interaction between primary and secondary components, as well as the flexibility of connections (sometimes they are of partial-strength, too) may significantly influence their response.

The buckling strength of these structures is directly influenced by the lateral restraining. Sensitivity to in-plane second-order effects, the
global imperfections, and the possible coupling of local and overall buckling may also influence their behaviour. According to EN1993-1-1[4], there are different imperfection shapes. On this purpose a preliminary analysis shall be performed in order to emphasize the importance of imperfection shape to be considered in the analysis.

In Fig. 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).

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[5] 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). The EN 1993-1 norm [4] 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 (γM0 = 1,00), given in [4] is still enough.

![Fig. 1: Imperfections recorded on site: a) assembling imperfection; b) manufacturing imperfection](image)

2 Analyzed frames and method of analysis

A series of frames of different spans and heights have been analyzed. The frames were designed to withstand the vertical loads and satisfy the ULS and SLS criteria. The frames have pinned base connections, tapered columns, hunched rafters and a pitch roof angle of 5° (Fig. 2). The length of the rafter hunch is 15% of the span in all the cases. The main dimensions of characteristic sections of frames are presented Table 1. The chosen dimensions are quite common in practical applications.

Both eigen-buckling (LEA) and nonlinear elastic-plastic considering geometrical nonlinearities (GMNIA) analyses had been applied [6,7]. The computation was performed with Abaqus 6.4 FEM program using Shell elements enabling for large plastic deformation. Joints were modelled using contact area-to-area elements in order to consider the real behaviour of the connections in the global analysis. The material behaviour was introduced by a bilinear elastic-perfectly plastic model, while S355 yield strength was considered in the analysis. Lateral
restrains provided in practice by purlins were considered. Moreover, in all these cases, it was modelled the restraining effect induced by longitudinal beams located at eaves and ridges.

![Fig. 2: Geometry of the analyzed frames](image)

**Table 1: Main dimensions of the analyzed frame**

<table>
<thead>
<tr>
<th>Code</th>
<th>Frame type</th>
<th>H [m]</th>
<th>L [m]</th>
<th>Dimensions (h \times b \times t_{w} \times t_{f} ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4x12</td>
<td>var4x12pin</td>
<td>4</td>
<td>12</td>
<td>tapered column: (250…600)<em>200</em>10*8</td>
</tr>
<tr>
<td>5x12</td>
<td>var5x12pin</td>
<td>5</td>
<td>12</td>
<td>tapered column: (250…600)<em>220</em>10*8</td>
</tr>
<tr>
<td>6x12</td>
<td>var6x12pin</td>
<td>6</td>
<td>12</td>
<td>tapered column: (250…600)<em>240</em>10*8</td>
</tr>
<tr>
<td>4x18</td>
<td>var4x18pin</td>
<td>4</td>
<td>18</td>
<td>tapered column: (350…700)<em>250</em>12*10</td>
</tr>
<tr>
<td>5x18</td>
<td>var5x18pin</td>
<td>5</td>
<td>18</td>
<td>tapered column: (350…700)<em>250</em>14*10</td>
</tr>
<tr>
<td>6x18</td>
<td>var6x18pin</td>
<td>6</td>
<td>18</td>
<td>tapered column: (350…700)<em>260</em>14*10</td>
</tr>
</tbody>
</table>

The lateral restraints are of 4 different types, as shown in Fig. 3 [8]. Types 2 and 3 models the purlin/sheeting effect, when the purlin can be connected with one or two bolts, respectively. Type 4 represents type 2 with an additional fly brace.

Type 1, the reference case, actually means no lateral restraints introduced by purlins. To simplify the computational model, in the analysis the lateral restraints had been considered axially rigid.

![Fig. 3: Types of lateral restraints](image)

Rafter-to-column and rafter-to-rafter connections are bolted as shown in Fig. 2. Vertical loads from permanent and snow actions were introduced at the purlin location (e.g. 1.2 m along the rafter). Both erection and manufacturing imperfections [8] were considered separately in analyses. The applied imperfections are presented in Fig. 4. Because the component elements of the structure are prone to out of plane buckling, the direction of the initial imperfections was considered accordingly. When the structures are modelled with shell elements beside out of plane buckling of the members (rafter and column) also initial twisting of the cross section can be observed. This represents the real case of the imperfect structure. At the first read the amplitude of the initial imperfections appears considerably high. It must be emphasised that according to EN 1993-1-4 [4], the initial imperfections shall considered also the residual stresses due to manufacturing process. In the present case the manufacturing process of the elements consist of continuous seam welding between web and flange.
Manufacturing imperfections (asymmetric out of plane bending and twisting of the rafter)

2IA
- 40 mm (12 m span frames)
- 60 mm (18 m span frame)

Assembling imperfections (asymmetric out of plane column displacement)

3IB
- 14 mm (4 m height frame),
- 17.5 mm (5 m height frame)
- 21 mm (6 m height frame)

Fig. 4: Imperfections considered in the FEM analyses

3 Results of numerical analyses

Nonlinear 3D elastic-plastic analysis and 3D eigenvalue analysis were performed to identify the elastic plastic-failure and critical modes of frames, respectively (Fig. 5 and Fig. 6). Correspondingly, critical and ultimate load multipliers were recorded and the results are presented, for all analyzed frames and out of plane restraining types (Fig. 7).
Analyzing these results one observes that lateral restraints influence the buckling shape and failure of the elements. When the structure is well laterally restrained (case 3 and 4 restraints of Fig.3), local buckling may develop prior to lateral-torsional mode. The critical local buckling load in cases of restraints types 3 and 4, sections is higher than the ultimate load obtained from elastic-plastic analysis, (see Fig. 7). We can say in this case that the structure may fail due to local plastic sectional buckling instead of elastic overall buckling.

For the frames described in Table 1, local and global imperfections were taken into account (Fig. 4). In Fig. 8 and Fig. 9, a comparison between perfect structure, initial bow (out of plane) imperfection and initial sway imperfection is presented. The applied lateral restraints are the ones shown in Fig. 3. Due to lack of space, herein only the results of 5x12 and 5x18 frames are illustrated, distinct for the two imperfection type (2IA, 3IB in Fig. 4)
The two types of imperfections considered in analysis influence significantly the bearing capacity of the considered structures (see Fig. 8 & Fig 9). 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 Fig 3) than in the case of well restrained ones (type 2 and 3 in Fig 3). Once more it was confirmed that 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.

4 Conclusion

A parametric study was performed in order to analyze the influence of two different types of imperfections (e.g. manufacturing imperfections characterized by initial bow imperfections, and erection imperfections characterized by initial sway imperfections) upon the behaviour of single storey frames made of elements with tapered web. The magnitude of the imperfections was considered as being equal with those prescribed in [4]. Within the performed analyses also lateral restraints, provided in practice by purlins, were considered.

It was noticed that the considered imperfections influences significantly the final capacity of the frame. The difference in the frame capacities is much higher in case of imperfect structure with low effective lateral restraint (type 1 and 2) than the imperfect structures well lateral restrained (type 3 and type 4). The difference between considered imperfections 2IA and 3IB is significantly low.

It was confirmed that in 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 authors gratefully acknowledge the financial support of “National University Research Council – NURC-CNCSIS-Romania” through the national research grant PN-II-RU-TE-2010-1/38.

References: