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THE INFLUENCE OF PURLIN-TO-BEAM CONNECTION STIFFNESS IN STRESS SKIN ACTION ON PORTAL FRAMES

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Abstract. *The paper presents the influence of purlin-to-beam connection stiffness, allowing to develop the stress skin action, in case of roof shear panels attached to pitched portal frames. Previous research evaluated the stabilizing effect on portal frames given by the roof diaphragm, formed by Z purlins and one layer of corrugated sheeting. Obtained results have shown the variation expressed by the load multiplication factor α_{cr} in sway type buckling of the frame and main components (columns and beams). The developed equivalent diaphragm models included all the calculated flexibilities, based on equivalent deformations, neglecting the influence of purlin-to-beam connection stiffness. Also, the eccentricities resulted from the type of different cladding systems, depending on the purlin and sheeting corrugation height should have an impact on the final results. The main purpose of this work is to separate the purlin-to-beam connection flexibility from the equivalent model and to evaluate the impact on frame lateral deformation and stability expressed by the load multiplication factor α_{cr} when purlin to rafter connection stiffness varies from flexible to a stiff one.*

1 INTRODUCTION

It is a well-known fact that the corrugated sheets, used as roof or wall claddings are often regarded as non-structural, secondary elements of a structure, this way disregarding completely the ability of the cladding system to transfer loads through their planar surface. This approach allowing for a more conservative and relatively fast structural design of frame structures. However, the last years showed a clear increase in research carried out on the diaphragm behavior of the cladding system, also called stressed skin action, and the problematics of including this effect into the design process.

Numerical calculations and experiments carried out by Szlendak et al. [1] on an industrial steel hall stated that the deformations in a structure subjected to horizontal loads can be reduced by 78% (sway) when considering the stressed skin diaphragm effect, applied only on the roof. Wrzesien et al. [2] describe in their paper laboratory tests conducted on cold-formed steel portal frames building in order to investigate the effects of stressed skin diaphragm action. They observed that the gable frames may be underestimated by as much as a factor of seven if the cladding system would be ignored. Since gable frames are usually designed to undertake half the load of intermediate frames, the authors recommend that the gable frames be designed at 1,7 times the force acting on a single bay between frames. Another method of quantifying the trapezoidal shear effect is described by Krahwinkel [3], where the total sheeting can be replaced by shear panels composed actually from n equivalent shear plates and rigid posts. By applying various formulas, specific dimensions for the shear plates can be determined and therefor they can be included in the shear panel as a way to consider the trapezoidal sheeting as a structural element. In the paper published by Dunai et al. [4], they obtained a higher scatter of results than expected due to the mounting process. Also they found that the eccentricities from the height of the purlins or corrugations, with or without wind bracing, have a significant influence on the overall response of the structure.

Previous research [5] investigates the diaphragm effect of the roof cladding system on portal frames. The study considered several corrugated sheet thicknesses, supporting types (two and four side), fastening methods (each through or every second only) and structural variations (frame number and bay changes), evaluating the portal frame structural performance in terms of load multiplication factor α_{cr}. Since roof diaphragm stiffness is fully transferred to the frame structure exclusively through the connection points, which are the purlin connections, present study focuses to evaluate the influence of the purlin to rafter connection type on the stressed skin effect transferred to the frame. A typical purlin-to-rafter connection used in practice was selected and only two sided supporting diaphragm solution was studied, since four sided supported diaphragm solutions are not usual, when trapezoidal sheeting's primary role is water proofing.

It seems that also calculation procedures for the evaluation of the stressed skin diaphragm flexibility shows some differences, the most significant being in case of flexibility term $c_{2,3}$, representing the contribution of purlin-rafter connection. The procedure published by Davies and Bryan [6] in 1982, the ECCS recommendations [7] published in 1995 and the Romanian code provisions NP 041 [8] issued in 2000 define in different ways the flexibility term $c_{2,3}$, depending on the global configuration of a two side supported diaphragm. The only difference is coming from the gable frame fixing, where connection elements between trapezoidal sheeting and the gable frame rafter may or may not exist. Due to fact that purlin-to-rafter connection in the analyzed diaphragms can have in some cases more than 50% contribution in global flexibility, it was expected to reach differences in analysis results, when purlin to rafter connection is separately modelled.

2 STRUCTURAL MODELS FOR THE ANALYSIS

To be able to compare the different analysis results, a similar portal frame configuration was built as in previous research [5]. The transverse frame was used with same geometry, but the 3D structural configuration was rebuilt in the current study by considering 4 bays (small span) and 10 bays (large span) for the roof diaphragm. For the small span, the stiffening effect of gable frame braces is almost fully transferred to internal frames through the roof diaphragm, in case of large span, the contribution of the roof diaphragm leads only around 10% redistribution of the loading for the middle frame. Only two simple load combinations were created for sake of simplicity: gravity loading (permanent and snow loads) and horizontal loading (permanent and side wind loading).

2.1 Structural configuration of the analyzed structures

The analysis was carried out on a single-storey structure, built up of portal frames, which had hinged column bases and fixed beams with haunched ends. The portal frames were connected by longitudinal pressure bars at the points where roof braces are connected to the rafter, hinged at both ends.

Characteristic dimensions of the considered structure (see figure 1):

- Span: 2 x 12.00 m
- Bay: 4 x 6.00 m and 9 x 6.00 m (5 and 10 frames)
- Length: 24.00 m and 54.00 m
- Eave height: +6.00 m
- Roof angle/pitch: 8 °

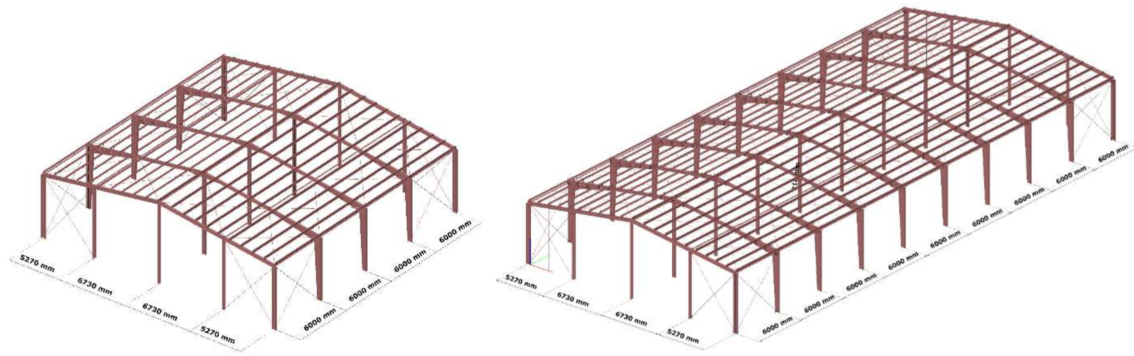


Figure 1: Considered building geometry in the analysis.

2.2 Applied loads on the structure

For sake of simplicity, only two simple load combinations were created, modelling typical gravity loading (permanent and snow loads) and typical horizontal loading (permanent and side wind loading). The following loads were applied on the structure:

- Permanent loads: g_k = 3.30 kN/m,
- Snow load on the roof: s_k = 7.20 kN/m,
- Wind load (side effect): w_k = 30.6 kN

The gravity loads are uniformly distributed on the frame rafter top flange (effect of loading position is considered), wind loads are applied as concentrated loads at the level of frame corner. Gable frame loading was reduced to half intensity. Two fundamental load combinations were created for the analysis, using the related partial safety factors, as recommended by EN 1991[9]: $1.35g_k+1.5s_k$ (combination 1) and $1.35g_k+1.5w_k+1.05s_k$ (combination 2).

2.3 Shear panel configuration

The cladding system acting as a diaphragm was composed of rafters, which were connected by purlins placed perpendicularly on top of them and corrugated sheetings, having the corrugation perpendicular to the purlins.

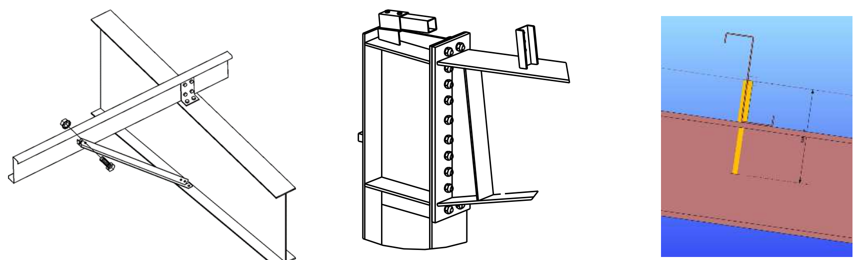
Only one type of fixing method of the corrugated sheet was considered in the analysis. The case when the diaphragm is fixed on two sides was denoted in previous research with 2L, and implies the fixing of the corrugated sheets only to the purlins. Purlin to rafter connection was modelled with stiff connector (shear panel flexibility included also the purlin to rafter connection flexibility) and with equivalent connecting element (shear panel flexibility excluded the purlin to rafter connection flexibility), equivalent connecting element was defined in such a manner, to reproduce the considered flexibility.

There is a discussion concerning the purlin to rafter connection flexibility, depending on the type of the connection. Usual connector types are presented in Figure 2 a and b, but only type a) have a specified flexibility in ECCS recommendation. Using very simplified analysis tools, it can be observed that such a connection for the directions indicated by ECCS recommendations specifies a 1.40 mm/kN flexibility for both directions of loading, but outward and inward loading of the purlin upper flange will result in very different flexibility, due to the distortion of the Z profile in case a (Fig. 3). For a U60x40x4 connector profile and Z200x2.5 mm purlin, the flexibility will vary from 0.2 mm/kN for inward loading direction up to 1.5 mm/kN for outward loading direction. Taking into account the opposite orientation of the purlins on the roof, it can be stated that for the same side loading due to wind action, the roof diaphragm separated by the ridge line will exhibit different flexibility. For the developed beam element, in the calculation models only the higher value of the purlin to rafter connection will be taken, very close to the value given by ECCS recommendations. Equivalent purlin connector (fig. 2 c) will be defined in such a way, to produce the same deformation for the length corresponding to the distance between rafter axis and Z purlin axis.

The analyzed shear panel's geometrical configuration is presented in table 1.

Table 1: Geometry of roof diaphragms.

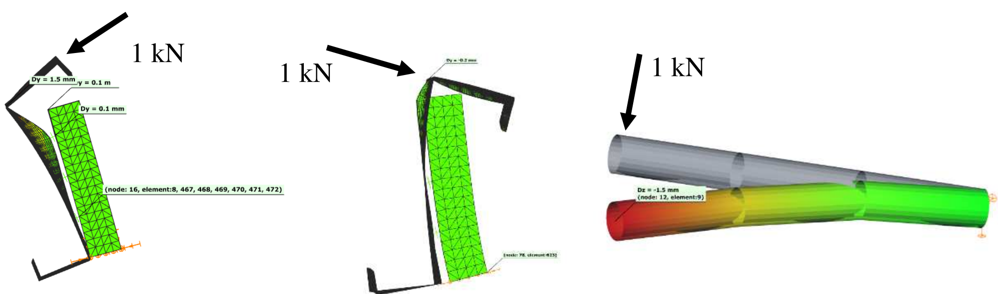
Diaphragm type	Span L [m]	Panel width a [m]	Panel length b [m]	Number of panels n
Roof	24	6	12.45	4 and 9



a. b. c.

Figure 2: Purlin to rafter connection

a. with flexible L profile, b. with rigid U profile, c. equivalent connector in the model



a. b. c.

Figure 3: Purlin to rafter connection a. outward load, b. inward load, c. equivalent connector

2.4 Description of the analyzed diaphragm system

The rafters having variable cross section along their length, with steel grade of S355J0. The used roof purlins were standard Lindab Z200 profiles, with a thickness of 2.5 mm, steel grade of S350 GD + Z275 and with corrosion protection by hot dip galvanizing layer of 20 µm on each side [10].

For the corrugated sheeting placed on top of the purlin two types of corrugated sheet plates were selected:

- Lindab corrugated sheeting LTP 45 with 0.5mm thickness [10];
- Megaprofil corrugated sheeting MP 85.280.1120 [11] with 1.25 mm thickness.

The geometrical characteristics of the utilized corrugated sheets are presented in table 2 and figure 4.

Table 2: Geometry of roof sheeting

Sheeting type	Height of corrugation h [mm]	Sheeting thickness t [mm]	Pitch of the corrugated sheet d [mm]	Length of one corrugation u [mm]	Wide flange l [mm]	Moment of inertia for the sheeting I_y [mm^4/mm]
LTP 45	45	0.5	180	231.437	77	161.06
MP 85	85	1.25	280	368.086	120	1448

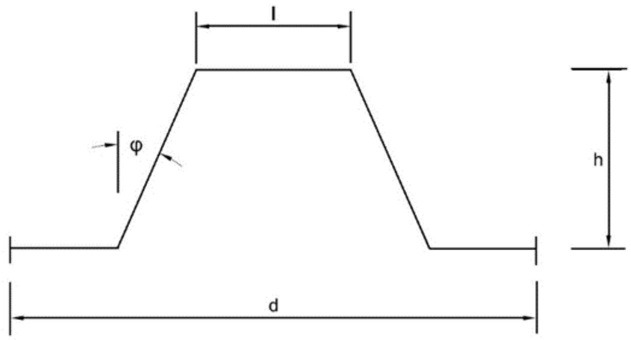


Figure 4: Geometrical characteristics of corrugated sheeting [10], [11].

2.5 Shear flexibility evaluation using [6], [7] and [8]

When computing the components of shear flexibility for a roof panel using [6, 7, 8], some differences can be noticed between the formulas for the following coefficients: shear strain in the corrugations ($c_{1.2}$), sheet to purlin fastener deformation ($c_{2.1}$) and purlin to rafter connection ($c_{2.3}$), all detailed in Table 3. By comparison, Davies' [6] approach is a more conservative one than the methods presented in NP041 [8] and ECCS [7]. For example: calculating the flexibilities for shear strain deformation of the corrugation ($c_{1.2}$) or the sheet to purlin fastener deformation ($c_{2.1}$) using NP041 [8] or ECCS recommendations [7], the values obtained will be at least 50% smaller than the same flexibilities calculated following Davies [6] formulas.

The most sensitive issue is the one concerning the purlin to rafter connection flexibility ($c_{2.3}$) because each method is different, although in every case the shear diaphragm is fastened only on two sides. Following ECCS [7] rules, even if the diaphragm is fastened on two sides only, gable shear connectors must be present, whereas in NP041 [8] and Davies' [6] approach, shear connectors on the gables are not mandatory (may or may not be present).

Table 3: Flexibility variation with different approaches

Calculation reference	$c_{1.2}$ [mm/kN]	$c_{2.1}$ [mm/kN]	$c_{2.3}$ [mm/kN]
NP041 [8]	$\frac{2*a*\alpha_2*(1+\mu)*\left(1+\frac{2*h}{d}\right)}{E*t*b}$	$\frac{2*a*s_p*p*\alpha_3}{b^2}$	$\frac{8*(n-1)}{n^2*n_p}*\left(s_{pg}+\frac{s_p}{\beta_2}\right)$
ECCS [7]	$\frac{2*a*\alpha_2*(1+\mu)*\left(1+\frac{2*h}{d}\right)}{E*t*b}$	$\frac{2*a*s_p*p*\alpha_3}{b^2}$	$\frac{4*(n-1)}{n^2*n_p}*\left(s_{pg}+\frac{s_p}{\beta_2}\right)$
Davies & Bryan [6]	$\frac{2*a*(1+\mu)*\left(1+\frac{2*h}{d}\right)}{E*t*b}$	$\frac{2*a*s_p*p}{b^2}$	$\frac{2}{n_p}*\left(s_{pg}+\frac{s_p}{\beta_2}\right)$

Symbols used in the formulas: a – dimension of shear panel in a direction perpendicular to the corrugations; α_2, α_3 – factors to allow for intermediate purlins, number of sheet lengths and sheet continuity; μ – Poisson's ratio; h – height of sheeting profile; d – pitch of corrugations; E – modulus of elasticity; t – sheet thickness; b – dimension of shear panel in direction parallel to the corrugations; s_p – slip per sheet/purlin fastener per unit load; p – pitch of sheet/purlin fasteners; n – number of shear panels in the length of the diaphragm assembly; n_p – number of purlins; s_{pg} – deflection of top of purlin at purlin/rafter connection per unit load.

To show the differences in the mentioned calculation methods [6, 7, 8] for several components of the diaphragm shear flexibility, Table 4 presents flexibility values obtained in the case of LTP45*0.5mm sheeting for the short type structure, composed of 5 frames, fastened on two sides with fasteners in every through, without gable fixing.

Table 4: Calculated flexibilities with different approaches

Calculation reference	Sheeting type	$c_{1.2}$ [mm/kN]	$c_{2.1}$ [mm/kN]	$c_{2.3}$ [mm/kN]
NP041 [C]	LTP 45*0.5 (5 frame structure)	0.006	0.001	0.211
ECCS		0.006	0.001	0.106
Davies & Bryan		0.018	0.002	0.282

ECCS recommendations does not give any formula for two side fastenings without gable fixing. For the structural analysis, the authors applied flexibility values calculated according to NP041 [8]. Entire shear panel flexibility in Table 5 was evaluated considering the given table value [8] or the computed value (Figure 3) for the purlin to rafter connection flexibility. Values registered in table 5 are valid for the short structure, composed of 5 frames. It must be noted that from the six different coefficients which sum up the shear flexibility of the panel, component $c_{2.3}$ has the greatest influence on the overall results (table 5).

Table 5: Considering the flexibility of rafter to purlin connection

Sheeting type	Thickness [mm]	Influence of flexibility of purlin to rafter connection					
		Rafter to purlin connection $c_{2.3}$ (given table value) [mm/kN]	Rafter to purlin connection $c_{2.3}$ (calculated value) [mm/kN]	Panel flexibility without $c_{2.3}$ [mm/kN]	Entire panel flexibility (with $c_{2.3}$ table value) [mm/kN]	Entire panel flexibility (with $c_{2.3}$ calculated value) [mm/kN]	Contribution of rafter-purin connection in global flexibility [%]
LTP 45	0,5	0,211	0,23	0,472	0,683	0,702	30,89%
	0,6			0,312	0,523	0,542	40,34%
	0,7			0,223	0,434	0,454	48,62%
MP 85	0,75			0,507	0,719	0,738	29,35%
	0,88			0,35	0,561	0,580	37,61%
	1			0,262	0,473	0,493	44,61%
	1,25			0,162	0,374	0,393	56,42%

Following paragraph will describe the finite element model built in Consteel software v.11 [12] using bar elements to model the effect of diaphragm on the framed structure.

3 EQUIVALENT SHEAR PANEL

The modelled components of the roof panels are the rafters, horizontal pressure bars, purlins and link type connections. As it was described in previous research, the equivalent shear panel model (Figure 5) was configured in such a way to estimate as close as possible the proper flexibility value of a shear panel, evaluated by NP041 procedure [8]. The model geometry includes the same number and position of purlin connections, as they exist in a real structure. Link elements have no geometric properties, only the axial stiffness which is equal with the shear panel flexibility resulted following the NP041 procedure. To keep the computer model as simple as possible, the previously simplified panel configuration was used with some minor improvement.

Components used in the model: 1 – marginal link type element; 2 – continuous purlin on 4 or more spans; 3 – X shape link element on each half of the panel; 4 – rafter of rigid end frame; 5 – horizontal bar pinned at both ends; 6 – linear fixed support; 7 – linear link element.

The stiffness of the panel was computed for two sides fastening, first with rigid purlin connectors, then equivalent purlin connectors were placed in the model, according to the principle presented in figure 3c. On the 3D calculation models, alternatively, rigid and equivalent connectors were used. Following paragraphs will summarize the results obtained using the developed models.

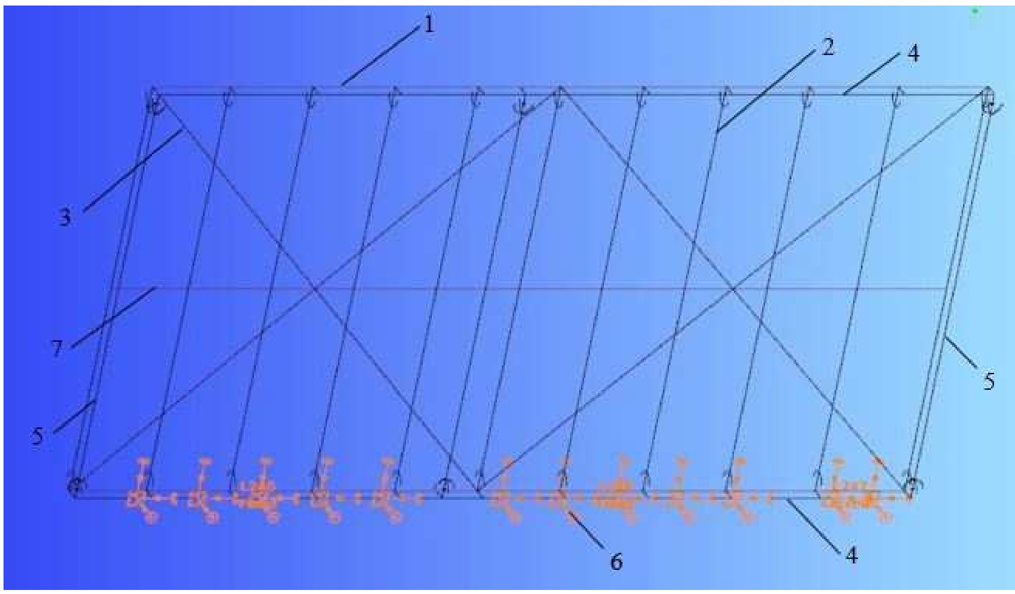


Figure 5: Equivalent shear panel of the roof

4 RESULTS OBTAINED WITH EQUIVALENT STRUCTURAL MODELS

Using the equivalent shear panels described before, 3D structural models were built, including the effect of roof diaphragm. Roof braces were excluded from the model, only the components modelling the shear panel were present. To compare the structural performance of these models, second order linear elastic buckling analysis was run, monitoring the column, beam and sway buckling of the transverse frames. Computed critical load multiplication factors are reported in table 6 and 7. The reference was considered the basic plane model of the frame (MB frame in table 6,7), excluding any stressed skin effect. The analysis of the MB frame is based on axially infinitely out of plane rigid constrains, where purlins, pressure bars and flange braces are connected to the frame. Calculation results were generated on short (5 frames) and long (10 frames) structures. In case of short structures, because internal frames are close to braced gable frame, the effect of roof diaphragm is more evident, since in case of long structures, those frames which are located far from the braced gable end, the stiffening effect of roof diaphragm is reduced. The analyzed models were realized with rigid purlin connectors (full diaphragm flexibility introduced in shear panel) and equivalent (EQ) connectors, where shear panel excludes the purlin to rafter connector flexibility, this effect introduced in the model with equivalent connector (figure 3c). The computed critical load multiplication factors are corresponding for gravity load combination (comb. 1), monitored transverse deformation of the frame corresponding to side wind action (comb. 2). Corresponding results for roof diaphragm with LTP45/0.5 and MP85/1.25 corrugated sheeting are reported in table 6 and 7 below.

Table 6: Results on analyzed structures, sheeting LTP45/0.5

Structure	$\alpha_{cr,column}$	$\alpha_{cr,rafter}$	$\alpha_{cr,sway}$	Deformation X [mm]
MB frame, plane analysis	4.30	11.43	8.85	66.00
10 Frames rigid connector	6.22	10.20	19.88	28.50
10 Frames EQ connector	3.80	10.22	19.56	29.10
5 Frames rigid connector	6.64	10.05	22.83	16.40
5 Frames EQ connector	4.13	10.20	22.74	18.00

Table 7: Results on analyzed structures, sheeting MP85/1.25

Structure	$\alpha_{cr,column}$	$\alpha_{cr,rafter}$	$\alpha_{cr,sway}$	Deformation X [mm]
MB frame, plane analysis	4.30	11.43	8.85	66.00
10 Frames rigid connector	6.22	10.20	22.20	26.40
10 Frames EQ connector	3.84	10.19	20.78	25.40
5 Frames rigid connector	6.64	10.05	22.83	15.50
5 Frames EQ connector	3.66	12.05	29.91	13.70

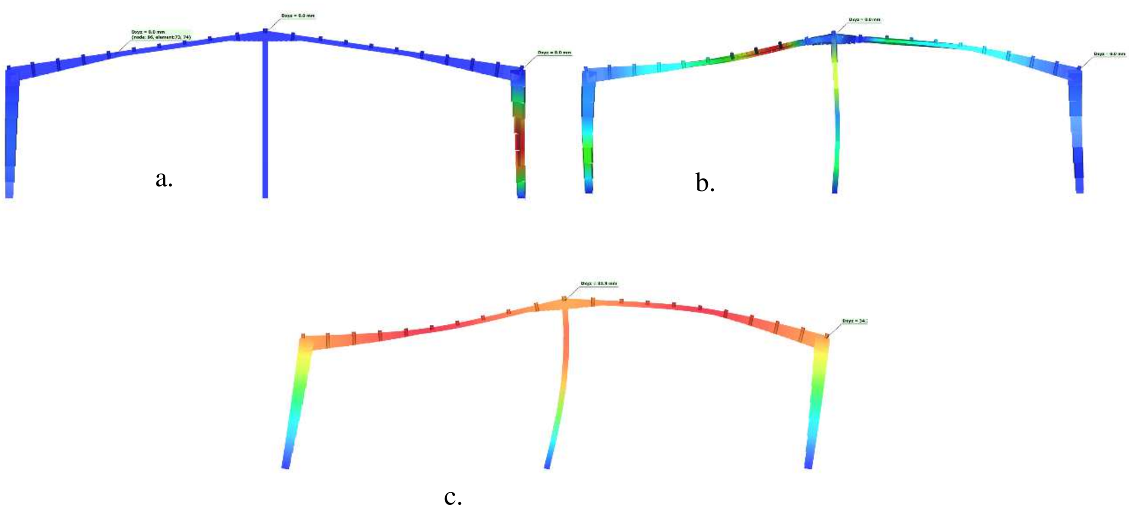


Figure 6: Reference frame. Monitored buckling modes:
a. External column buckling, b. rafter buckling, c. sway buckling of the transverse frame

5 DISCUSSIONS

In the first instance, analysis results highlight the increase of lateral stiffness of the frames. Compared with the simple portal frame, which lateral stiffness is given only by the rafter to column connection stiffness, application of roof diaphragm on the structure has a direct effect of lateral stiffness increase. This fact is confirmed by the drop down in horizontal deformations of the frame with more than 50% and there is an evident increase of critical load multiplication factor for sway buckling of the frame (more than double in all the cases). It is also interesting to mention the influence of purlin to rafter connection stiffness on the lateral deformations of the transverse frame. In case of LTP45/0.5 sheeting (reduced diaphragm stiffness), where purlin to rafter connection stiffness contribution in the global stiffness of the diaphragm is around

31%, rigid connector gives lower and equivalent connector gives higher lateral deformations in the analysis. In case of MP85/1.25 sheeting (increased diaphragm stiffness), where purlin to rafter connection stiffness contribution in the global stiffness of the diaphragm is around 57%, rigid connector gives higher and equivalent connector gives lower lateral deformations in the analysis.

Another interesting impact in the analyzed models is related to the column buckling. Rigid purlin to rafter connector increase the stability of the marginal columns ($\alpha_{cr,column}$ is higher than in case of reference frame) since equivalent purlin to rafter connector decrease the stability of the marginal columns ($\alpha_{cr,column}$ is lower than in case of reference frame). In all the analyzed cases, frame rafter behavior is a stable one, all the registered stability losses are with the purlin system. Pure flexural or torsional-flexural stability loss shapes can be reached at higher level of the critical load multiplication factor.

It is also important to mention that equivalent connectors are able to introduce the eccentricity effect coming from the purlin height. As it was demonstrated in previous research, increase of eccentricity have a direct effect of increasing diaphragm flexibility. Anyway, the equivalent connector can introduce in the calculation model only the purlin-rafter eccentricity, the eccentricity resulted from the corrugated panel height is not yet modelled. Loading of the frame in the analysis was done direct to the rafter upper flange, but taking into account that loaded surface is the corrugated sheeting, additional negative effects can result from cumulated eccentricities.

6 CONCLUSION

The influence of purlin-to-beam connection stiffness has been analyzed, allowing to develop the stress skin action, in case of roof shear panels attached to pitched portal frames. Obtained results express the variation by the load multiplication factor α_{cr} in sway type buckling of the frame and main components (columns and beams). The developed equivalent diaphragm models include all the calculated flexibilities, based on equivalent deformations, taking into account the influence of purlin-to-beam connection stiffness. Obtained result on models with stressed skin action showing an increased lateral frame stiffness, obtaining less lateral deformations and an increased sway buckling resistance. As a summary, the following conclusions can be drawn:

1. While a simple plane frame is predisposed to buckle, being classified as sensitive to second order effects ($\alpha_{cr,sway} \leq 10$), introducing in the 3D model only the roof diaphragm effect, the lateral stiffness of the frame is increasing, exhibiting much higher sway stability;
2. The increase of lateral stiffness is confirmed by the drop down in frame corner displacement: while simple plane frame exhibits 65 mm displacement under side loading, in the 3D structural configuration with 5 frames under similar loading conditions, the internal frame deformation with LTP45/0.5 roof sheeting is around 18 mm and MP85/1.25 roof sheeting is 13,70 mm. This reduction confirms the 78% reduction measured on a real structure by Szlendak [1], having similar structural configuration;
3. Introducing in the 3D structural model the purlin to rafter connection flexibility using equivalent connector, rafter stability is not affected, but lateral column exhibits lateral torsional buckling with higher critical load than reference frame column, if purlin to rafter connector is modelled as rigid and with lower critical load than reference frame column, if purlin to rafter connector is modelled with equivalent stiffness. The stiffness

of the frame corner in the analysis was neglected, following research intending to study the effect of corner stiffeners type as well.

4. The scattering in column critical load multiplication factor seems to be sensitive to the purlin connector stiffness and less sensitive to the roof diaphragm stiffness. It is supposed to have similar analysis results with different roof diaphragm stiffness;
5. The purlin to rafter connection flexibility is highly depending by the type of used connection. Using very simplified analysis tools, it was found that analyzed purlin to rafter connection flexibility takes very different values, depending by the directions of loading. Due to this, the behavior and deformations exhibited in reality by such diaphragms can be different compared with evaluation results following NP041 [8] or ECCS rules [7];
6. The procedure published by Davies and Bryan [6] in 1982, the ECCS recommendations [7] published in 1995 and the Romanian code provisions NP 041 [8] issued in 2000 define in a different way the flexibility term $c_{2,3}$, depending on the global configuration of a two side supported diaphragm, leading to obtain flexibility values from simple to double.

Conclusions would be limited to the studied types of diaphragms. Research will continue to extend the conclusions for diaphragms realized with other type of corrugated sheeting and validation of equivalent models by laboratory testing.

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