STRESSED SKIN EFFECT ON THE ELASTIC BUCKLING OF PITCHED ROOF PORTAL FRAMES

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STRESSED SKIN EFFECT ON THE ELASTIC BUCKLING OF PITCHED ROOF PORTAL FRAMES

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Abstract. *The paper presents the influence of the diaphragm effect on the behavior of pitched roof portal frames, having Z purlins and corrugated sheeting as cladding. The paper highlights the stabilizing effect in terms of α_{cr} on portal frames by taking into account the lateral constraints ensured by a typical cladding system – Z purlins with one layer of sheeting panels. The purpose of the paper is to make a comparison between the simplified design model of a portal frame, where the supports simulating the purlins are considered with infinite axial rigidity and a portal frame design model where the calculated stiffness of the cladding for the lateral supports is introduced manually. The obtained results highlight the importance of the diaphragm effect and refer to the variation of the load multiplication factor α_{cr} for main structural elements. The fundamental objective of this research is to develop a relatively fast checking procedure, easy to use in the current design process, by including the diaphragm stiffness in the analysis of the pitched roof portal frame. Using Abaqus, simplified calculation procedures are validated by complex FEM models.*

1 INTRODUCTION

Even since the 1950's, the idea that systems assembled from corrugated sheets, used as roof or wall claddings, which were properly fixed, in addition to their ability to undertake perpendicular loads to their plane, can also undertake loads acting in their planar surface (Johnson [1]). Loads can be a result of wind action, earthquake and interaction between the frames' claddings or of the diaphragm behavior for certain types of roofs (Davies and Bryan, 1982 [2]). In all cases, loads applied in the sheeting plane lead to stresses. The resulting stressed membrane is usually defined as a diaphragm. In general, corrugated sheets are subjected to shear forces, while the axial efforts are undertaken by the elements of the transverse frames. This membrane or diaphragm brings for the structure added strength and stiffness and can be used to stabilize structural elements. In Europe, the design methodology using this type of action is called “stressed skin design” (Bryan, 1973 [3]) – the diaphragm effect. The shear or diaphragm panel refers to one or several corrugated sheeting’s separated by structural elements, being part of the shear diaphragm.

In reality, the diaphragm effect is present in a structure, whether it was taken into account or not. Economic studies carried out in Europe by organizations such as the European Convention for Constructional Steelwork (ECCS) or the Constructional Steel Research and Development Organization (CONSTRADO, 1976 [4]), indicate that it can be saved up to 10% of the total cost of the steel structure if the diaphragm effect is taken into account.

Since the 1980's, in Europe, general prescriptions can be found for the configuration of diaphragms in order for them to be designed more efficiently. Bryan and Davies (1982, [2]) prepared recommendations for the size of the panels and give design rules for the shear and seam connectors and the connections between purlins and bearing structure. Their book includes assembling regulations also. According to these studies, the trapezoidal sheeting is preferred instead of the sinus shape and has also better documented references.

Regarding the testing of the shear diaphragms and shear panels, among the first well documented procedures were the ones undertaken by the American Iron and Steel Institute (AISI) in 1987 [5]. Full scale tests were done both for regularly in plane shaped models and irregularly shaped ones. Load bearing capacity and flexibility for sheet to sheet and sheet to bearing structure fastenings are presented in the ECCS publications (1978, 1984).

Although a 1977 version of the “European Recommendations for the application of Metal Sheeting acting as a Diaphragm” was published, an improved version was issued in 1995.

As recommended in the ECCS publication [6], the design of structures taking into account the diaphragm effect involves the cladding structure as an integrating part of the main load bearing structure and designing it as a diaphragm subjected to shear force, which is mainly used to increase structural stability. Romanian code provisions [7] mainly follow the ECCS recommendations [6].

The primary role of roof and wall cladding systems is to ensure water and air tightness for the building, while the diaphragm effect transforms them into main structural elements. This conversion of secondary structure into primary structural components must keep focus on the usual cladding details, with the typical sheeting thicknesses and the applied sheet-to-purlin screws and seam fasteners. Starting from these input data and accepting that supplementary measures will generate higher cladding costs, the following questions can be raised:

- How relevant is the type of trapezoidal sheeting in order to account for the diaphragm effect?
- How much is the final stiffness of the diaphragm panels influenced by the way of fastening the corrugated sheeting?
- What is the gain in load carrying capacity if the structure is stabilized by the cladding system instead of neglecting this effect – expressed in percentages?
- Is it relevant to have a structural design in which the steel sheeting is considered to be acting as a diaphragm, taking into consideration that in most design cases the cladding is not considered as having a structural role?

2 RESEARCH OBJECTIVES

The paper presents the diaphragm effect on the behavior of pitched roof portal frames, having Z purlins and corrugated sheeting as cladding. The idea of this study emerged from economical design principles and from the desire to quantify the structural safety reserves using a more detailed analysis of structural elements.

Taking the example of an existing structure, a steel framed industrial hall with simple geometry and trapezoidal sheeting as cladding, the major goal was to find the influence of the roof sheeting acting as diaphragm over the main steel structure, highlighting how the structure works together with the cladding in order to undertake vertical and horizontal loads and calibrating an automatic computation procedure in a dedicated environment, appropriate in structural design.

There are several analysis parameters to be considered, deriving the following specific research objectives:

- type of trapezoidal sheeting – comparison between the different types of steel sheeting which can be used for the proposed structure, having as variables the trough height and material thickness;
- fastening of the trapezoidal sheeting and purlins – every trough fastened or alternate troughs fastened;
- supporting of the shear panels – two or four sides fastening of the shear panel.

The following chapters include the assumptions and analysis criteria, computations and obtained results together with their graphical interpretation.

3 STRUCTURAL CONFIGURATION USING A CASE STUDY

The above mentioned parameters monitored in this research have been analyzed using a case study on an existing structure located in Oradea, Bihor County, Romania [8]. The focus of this study is to quantify the stressed skin effect in order to obtain better design results for pitched roof portal frames with corrugated sheeting as cladding.

3.1 Geometrical and structural configuration of the analyzed structure

The object of the study consists of a single storey steel structure. The loadbearing steel structure is made of portal frames with hinged column bases and transverse haunched beams fixed to the columns and horizontal rulers hinged at both ends, placed between the frames. Initial design of the structure neglects the stressed skin effect, where X roof and wall bracings were provided.

The original structure has the following characteristic dimensions:

- Span: 2 x 12.00m
- Bay: 15 x 6.00m
- Length: 90.00m
- Eave height: +6.00m
- Roof angle/pitch: 8˚

The building consists of the office area and the production area (Figure 1). The office area extends along one bay (6.00m) and over the whole span of the hall (24.00m), while the production area occupies the rest of the space. The office area has two stories separated by a steel floor in dry solution, as shown in Figure 2. Beams situated on the perimeter and those between the columns are fixed at both ends, while the rest of the remaining secondary beams are hinged.

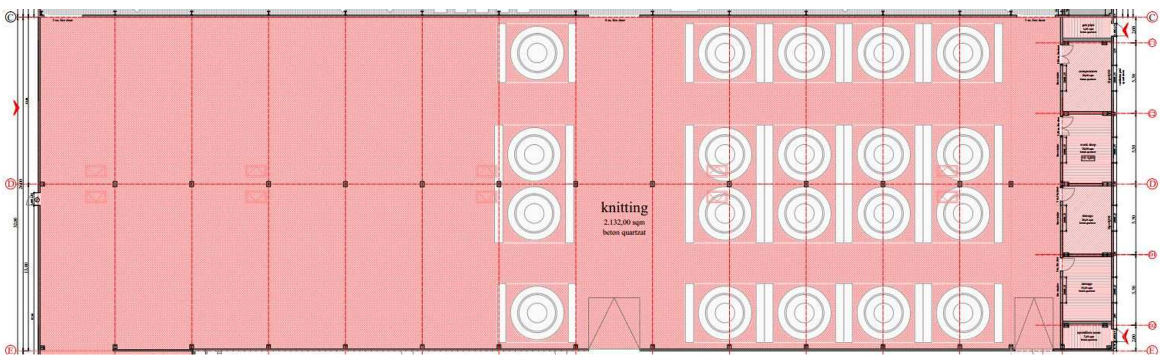


Figure 1: Architectural plan of the analyzed building [8]

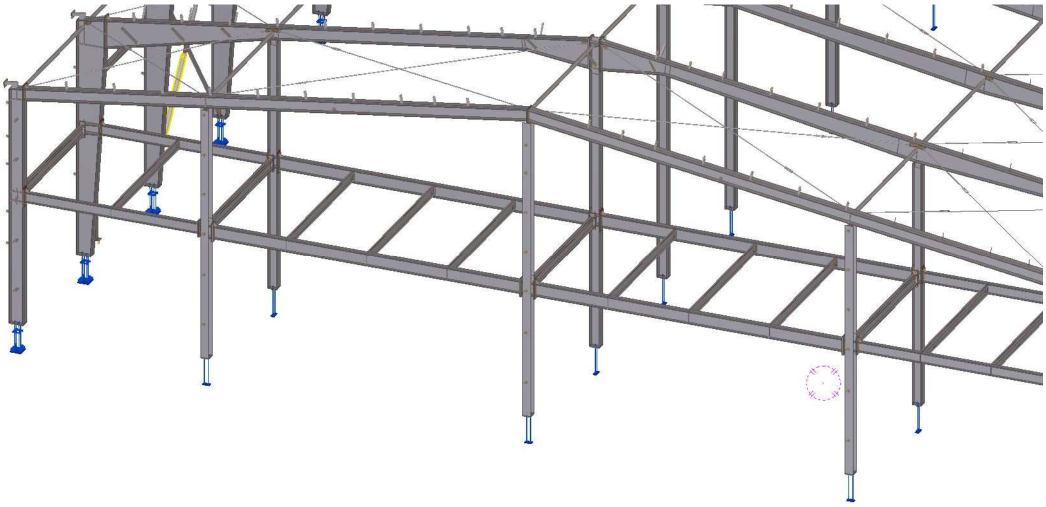


Figure 2: The office area in the modeling stage [8]

3.2 Relevant loads for the structure design

The loadbearing structure of the building is subjected to the action of its own weight, to live loads, technological loads, climatic loads and seismic action respectively. These loads were determined according to the Romanian regulations, in force at the time of designing the structure.

The loads were considered as follows:

- Permanent and technological loads g=0,50 kN/m^2
- Live load on the intermediary floor: q=3,0 kN/m^2
- Distributed snow load on the roof: $s_{0,k}$=1.5kN/m^2 – characteristic value for snow load on the ground
- Distributed wind load: w_{ref}=0.50 kN/m^2 – reference pressure

Seismic loads determined according to Romanian seismic code for the structure's specific location γ_1=1.0 (importance class 3), a_g=0.15g – peak ground acceleration, T_c=0.7s – corner period. The behavior factor in this particular configuration was considered for low dissipative structures, q=1.5.

For the detailed analysis and design checks of the structure according to EN1993-1-1, EN1993-1-3 and EN1993-1-8, available with NAD [9], ConSteel 8.0 finite element software has been used. Thus, calibration of the calculation methodology to account for the stressed skin action has been done in the same software environment.

3.3 Cladding acting as a diaphragm system

According to ECCS - “European Recommendations for the Application of Metal Sheeting acting as a Diaphragm – Stressed Skin Design, 1995” [6], the cladding systems can act as diaphragms, which represent a plane assembly made out of panel sheets, purlins and rafters. The sheeting panels are fixed only to the purlins (case of diaphragms fixed only on two sides) or to purlins and rafters (case of diaphragms fixed on four sides). The side overlaps are fixed together with seam fasteners. The diaphragm may be composed of a single panel or multiple panels.

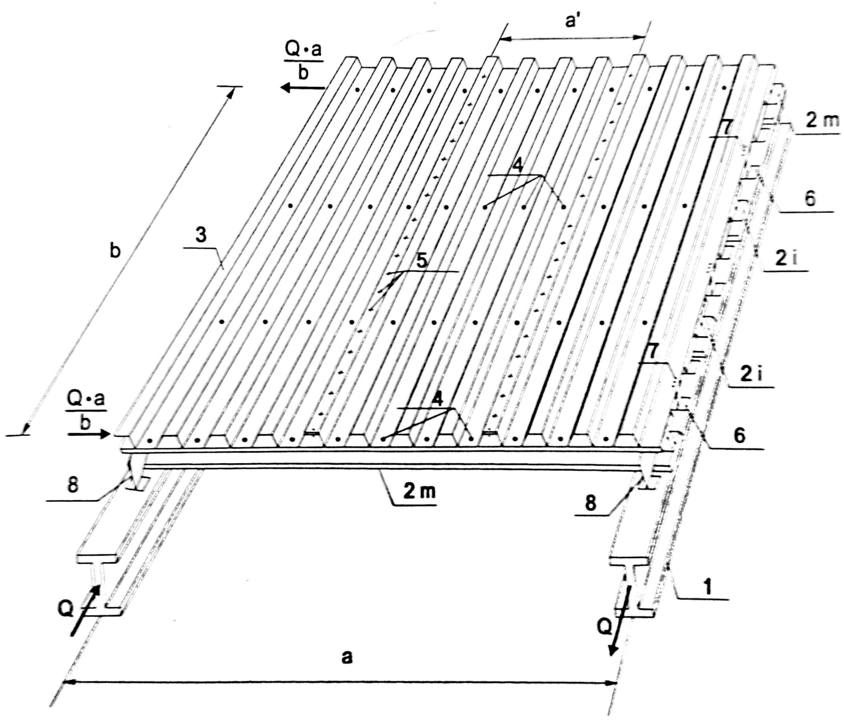


Figure 3: Typical shear panel [7]

Legend: a = dimension of the shear panel in a direction perpendicular to the corrugations; a` = width of a sheeting panel; b = dimension of the shear panel in a direction parallel to the corrugations; 1. rafter – main beam; 2m. edge purlin – secondary beam; 2i. intermediate purlin – secondary beam; 3. corrugated sheeting panel; 4. sheet/ perpendicular member fastener; 5. seam or side lap fasteners; 6. shear connector; 7. sheet connector fasteners; 8. purlin/ rafter connection.

Due to various openings which can weaken the wall diaphragms, the present study focuses mainly on the roof diaphragms. Their geometry is described in Table 1.

Table 1: Geometry of roof diaphragms

Diaphragm type	**Span L [m]**	**Height b [m]**	**Panel dimensions [m]**		**Number of panels**
			a	**b**	**n**
Roof	90	12.40	6	12.40	15

The components of the roof shear panels considered in the models include:

- frame rafters as main beams;
- roof purlins as secondary beams;
- corrugated sheeting;
- shear connectors and seam fasteners.

3.4 Description of the analyzed diaphragm system

In case of roof shear panels, the frame rafters are not at the same level with the diaphragm. The fastening of the diaphragm can be done either on two sides to the secondary beams or on four sides: both to secondary and main beams using connectors.

The cross sections of the main beams are composed from welded plates of various dimensions, with the steel grade of S355J0.

The secondary beams or purlins are standard Lindab profiles Z 200 with a section thickness of 2,5mm on the first bay and 1,5mm on the rest of the bays. The purlins' material is galvanized steel S355 with corrosion protection by hot dip galvanizing layer of 40 μm [11].

The corrugated sheeting is acting as roof cladding and is placed on top of the purlins. For the stressed skin action, two types of sheeting (Figure 4, Table 2) available in Romanian market were compared, each with several thicknesses:

- Lindab corrugated sheeting LTP45 with 0.5, 0.6 and 0.7 mm thickness;
- Megaprofil corrugated sheeting 85.280.1120 [12] with 0.75, 0.88, 1.00 and 1.25 mm thickness.

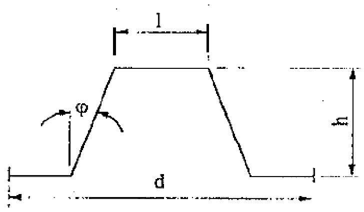


Figure 4: Geometrical characteristics of corrugated sheeting [7]

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]**	**Moment of inertia for the sheeting I_y [mm^4/mm]**
LTP 45	45	0.5	180	231.437	161.06
		0.6			196.56
		0.7			232.44
85.280.1120	85	0.75	280	368.086	774.5
		0.88			956.5
		1			1125
		1.25			1448

The considered fasteners for sheet/rafter connections are SD5 type with shear strengths according to SFS Intec Catalogue [13] and the seam fasteners are type SL2-S-4 according to the same catalogue.

4 EQUIVALENT MODELS AND ASSUMPTIONS.

The calculation of the roof diaphragm strength and stiffness was performed according to ECCS recommendations [6] and "Manual of stressed skin diaphragm design" - Davies and Bryan [2], which have also been followed by the Romanian code – NP 041 – 2000 [7]. The studied specific case refers to a roof diaphragm without openings, supported at both ends and composed of several shear panels having the sheeting perpendicular on the diaphragm's span.

According to the prescriptions imposed by the above mentioned codes, the study was conducted for a single storey structure with steel portal frames subjected to vertical and horizontal loads, having trapezoidal sheeting as roof cladding. The structure was considered as a set of elements which work together, having rigid end frames and flexible intermediate ones, with pinned column bases and continuous eave joints.

4.1 Design criteria according to ECCS recommendations

According to ECCS provisions [6] the final diaphragm strength is obtained considering the following failure modes: a. sheet tearing along a line of seam fasteners, b. sheet tearing along a line of shear connector fasteners, c. sheet tearing in the sheet/purlin fasteners, d. end collapse of the sheeting profile, e. shear buckling of the sheeting, f. failure of the edge member in tension or compression. The preferred modes of failure are a and b. The minimum capacity of these failure modes will define the ultimate loadbearing capacity Q_{ult}.

Above mentioned regulations also implement the procedure for calculating the shear panel flexibility. The total shear flexibility “c” of a panel represents the sum of the separate component shear flexibilities due to the following deformation modes: profile distortion, shear strain in the sheet, slip in the sheet/purlin fasteners, slip in the seam fasteners, slip in the sheet/shear connector fasteners, purlin/rafter connections (in the case of the sheet fastened to the purlins only), axial strain in the longitudinal edge members.

Following the formulas specified in ECCS provisions [6], both the flexibility and stiffness of each individual roof shear panel was computed manually. Obtained results are reported in chapter 5.

4.2 Calibration of a design procedure: equivalent static models

In the current design practice, 3D analysis of the structure is performed in most situations, without taking into account the frame stabilizing effect of the sheeting. This case study presents the influence of trapezoidal sheeting acting as diaphragm over an existing single storey steel structure with portal frames. Several structural models were made, leading to results in terms of structural performance with and without taking into consideration the diaphragm effect of the roof cladding.

The considered static design models are: the gross model (MB), transition model (MT1, MT2, MT3) and equivalent models (ME1...7), as it is defined below.

4.2.1 Gross model

This configuration represents a 3D structural model which includes the main bearing elements:

- transverse frames with pinned base joints
- longitudinal bars at the eave and ridge joint and, also, in the middle of the rafter;
- roof and walls bracing systems;
- flange braces;
- roof purlins positioned only where flange braces are necessary.

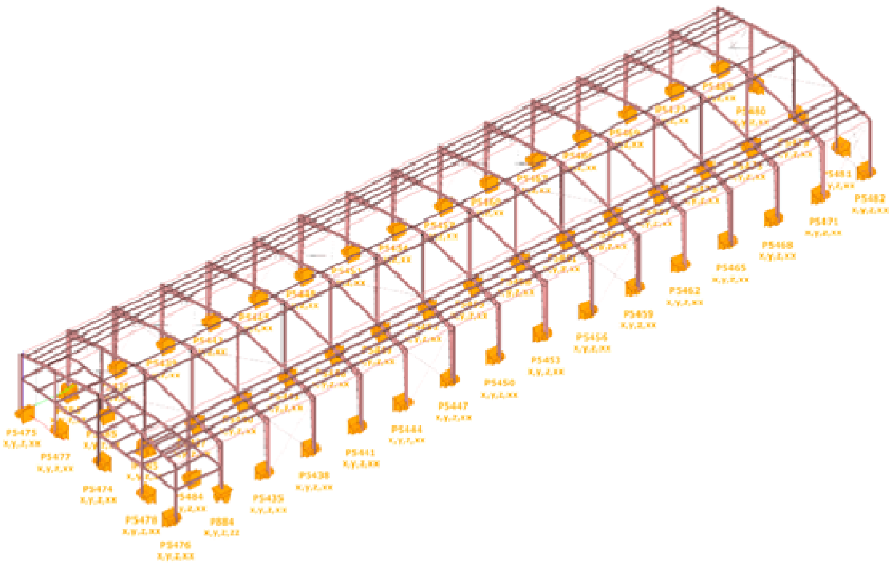


Figure 5: Gross model (MB) – 3D geometry

4.2.2 Transition models

Transition model has as a starting point the gross model, including the entire roof purlins and side rails system. This represents an intermediate stage in which a part of the cladding system is included, but still neglecting the stiffening effect of the trapezoidal sheeting.

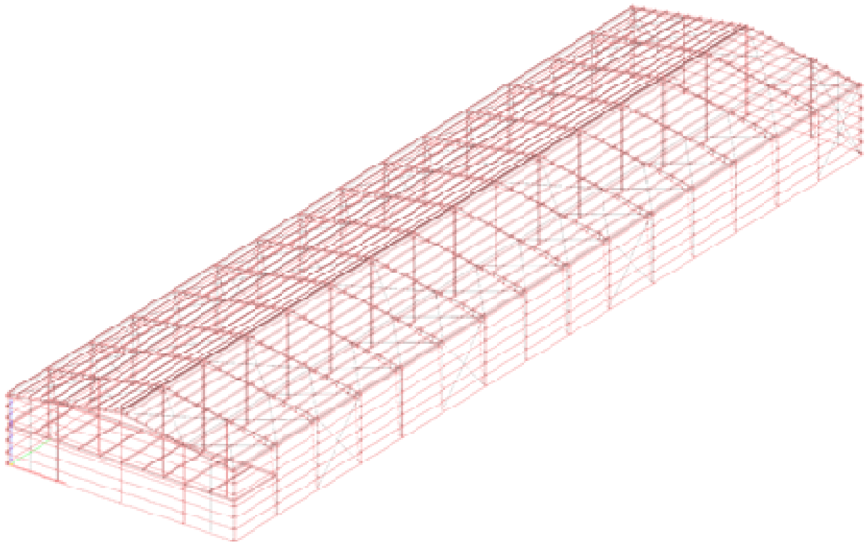
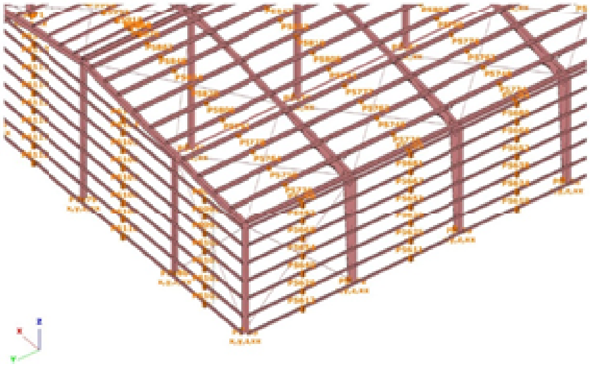


Figure 6: Transition model – MT 1

Starting from the idea of setting up an intermediate model including the purlin system and side rails, three variations of the same model were built, namely:

- MT 1 – gross model improved with purlins and side rails (Figure 6);
- MT 2 – MT 1 model to which supplementary supports against axial rotation were added on the purlins and side rails (Figure 7a);
- MT 3 – to MT 2 model more supports against axial rotation were added together with a system of links to induce a simultaneous movement of the side rails and purlins (displacements and distances between the elements remain constant), without taking into account the stiffness of steel sheeting (Figure 7b)



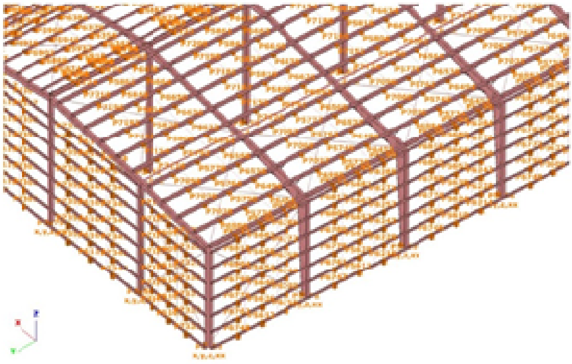


Figure 7: Transition models MT 2 (a - left) and MT 3 (b - right)

4.2.3 Equivalent models

Compared with the previous models, the equivalent models (Figure 8) includes the stiffness of the roof cladding panels. This is possible by developing equivalent models for the shear panels in the initial stage and then applying them on the 3D model. The step by step procedure is described in the following subchapters.

Depending on the type of trapezoidal sheeting used in panel analysis, the configured models are grouped in the table below.

Table 3: Equivalent model configuration

Cladding type	Sheeting type	Sheeting thickness t [mm]	ME 1	ME 2	ME 3	ME 4	ME 5	ME 6	ME 7
Roof	LTP 45	0.5	x						
		0.6		x					
		0.7			x				
	MP 85.280.1120	0.75				x			
		0.88					x		
		1.00						x	
		1.25							x

The equivalent models analysis is performed depending on the support type of the roof diaphragm: on two or four sides fastening.

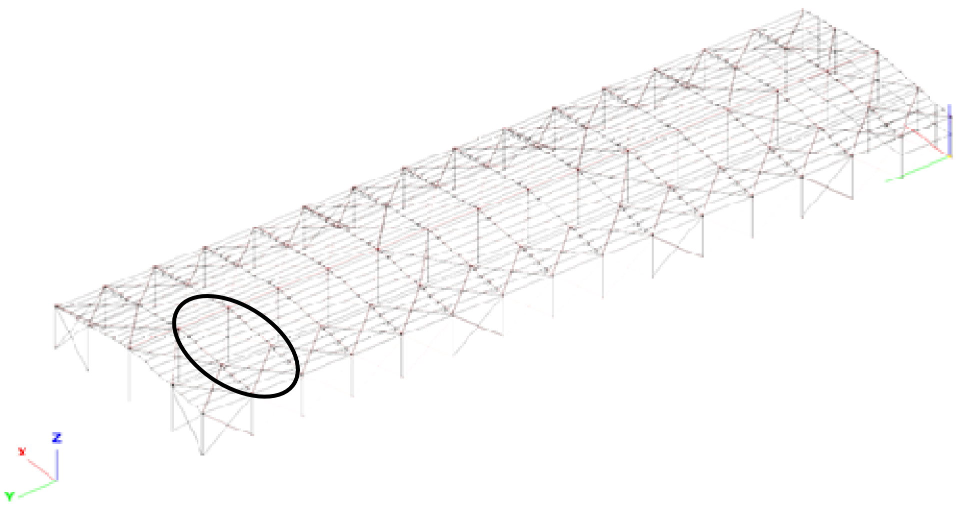


Figure 8: Equivalent model with marked shear panel

4.2.4 Shear panel modeling

Based on the calculation procedure developed by ECCS [6] and also included in the Romanian code NP 041 [7], it is necessary to calculate individual shear panels and then create a shear panel model in the static design software in order to build the equivalent 3D model of the structure. The stiffness of the roof shear panels were computed individually for each of the fastening methods, on two and on four sides.

The stiffness was introduced in the structural modeling considering aspects like efficiency and accuracy in order to find an optimal solution when creating the shear panel.

To obtain a shear panel model in the static design software, several configurations with different boundary conditions were made. Link type connections were used at this stage, with the stiffness introduced according to the manually obtained values, in kN/mm. Then, on each panel a unit force in kN was applied to determine the displacement in the load direction. As a result, the panel flexibility values were obtained and then compared to the manually computed values. The selection of the proper panel model was made based on the above mentioned aspects. The configuration stages of the static models are presented in the Figure 9.

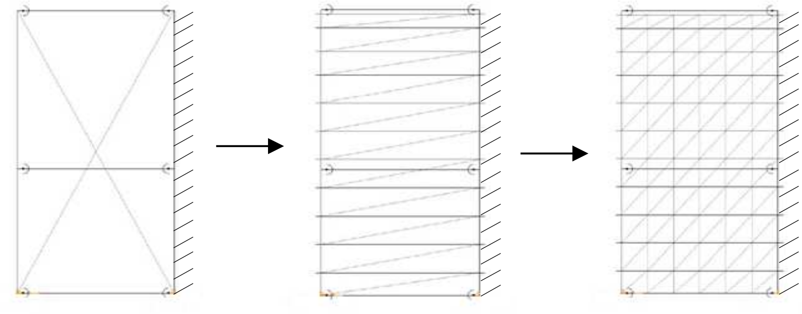


Figure 9: Stages in the configuration of the roof panel models

4.2.5 Proposed equivalent roof shear panels for analysis

The components of the roof panels are: the rafters, horizontal rulers, purlins and link type connections. The final model (Figure 10) was configured as to obtain the proper flexibility value and to be able, from the modeling point of view, to include all the existent geometric elements. It has to be mentioned that the link elements have no geometric properties, only the computed panel stiffness, used as axial stiffness.

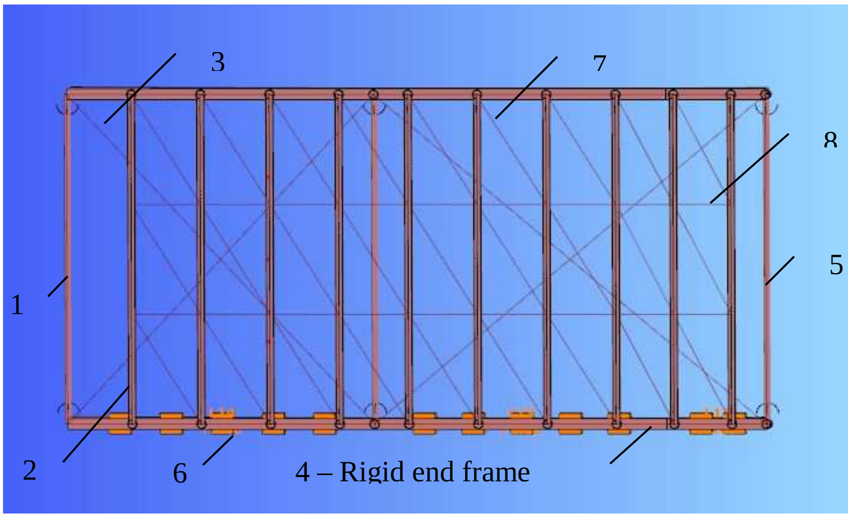


Figure 10: Characteristic roof panel with introduced links in ConSteel software (6.00x12.40m)

1 – perimeter link type element ; 2 – continuous purlin on 4 or more spans; 3 – X shape link element on each half of the panel; 4 – rafter; 5 – horizontal bar pinned at both ends; 6 – linear fixed support; 7 – diagonal link element; 8 – linear link element.

The stiffness of the panels was computed for two and four sides fastening and the main difference between the two is represented by the flexibility of sheeting – rafter fastening (through shear connectors). The static model configurations of the panels are considered the same, noting that all the link elements have been introduced in the software with specific boundary conditions. The specific boundary conditions refer to implementing the computed panel stiffness for each panel as axial link stiffness according to the procedure described in [6] and [7] (Figure 13).

4.3 Combined action between the roof diaphragm and transverse frames according to ECCS recommendations and Romanian code NP 041

3D combined action of transverse frames, achieved by roof trapezoidal sheeting diaphragm, performs only if the structural configuration leads to a differentiated lateral displacement of the transverse frames and is more active if the ratio $\Psi = c / \delta$ between the diaphragm flexibility c and transverse frame flexibility δ is smaller. The significance of c and δ flexibilities in order to compute the Ψ ratio is presented in Figure 11.

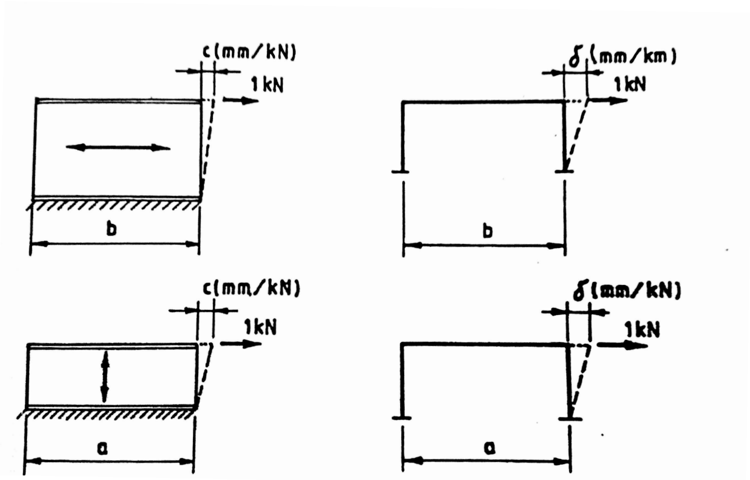


Figure 11: Graphical representation of flexibilities according to [7]

a – shear flexibility c of the panel; b – δ flexibility of transversal frame

In the analysis software, the flexibility of the frame results by measuring the displacement in mm when applying a unit horizontal force in kN in the eave joint, in the direction of the transverse frame.

Combined action analysis for complex structures with more spans is performed following the ECCS procedure [6]. The modeling of the roof shear panels made by trapezoidal sheeting was introduced in the analysis software in accordance with the panel model presented in Figure 10.

5 RESULTS AND DISCUSSIONS

Several cases of the diaphragm effect were analyzed for the presented single storey steel structure. The results were compared in terms of loadbearing capacity and flexibility for common types of trapezoidal sheeting used for roof cladding.

Parameters taken into account are:

- trapezoidal sheeting type: thickness, trough height;
- fastening type: each trough and alternate troughs;
- method of supporting: on two or four sides.

According to the obtained results in terms of combined action between the roof cladding and structure, it is relevant to take into consideration the diaphragm effect ensured by the system of purlins and trapezoidal sheeting. The results are expressed graphically in terms of load amplification factor α_{cr}, for the dominant combination (permanent + technological loads with snow loads) which designs the structure.

The results obtained from the analysis of diaphragm panels were grouped according to:

- load bearing capacity of the diaphragm;
- flexibility and stiffness of the diaphragm.

5.1 Bearing capacity for the roof diaphragm for all studied sheeting types, according to ECCS recommendations

For the roof diaphragm system, the analyzed cases are the ones in which the sheeting is fastened to the beams in every trough and in alternate troughs. The roof diaphragm is considered to be supported on two sides and on all four sides. Thus, depending on the analyzed case, the ultimate bearing capacity of the diaphragm depends on either the strength of the seam fasteners (case 4L) or on the strength of sheet/ purlin fasteners (case 2L). The results are presented in Figure 12, depending on the sheeting thicknesses.

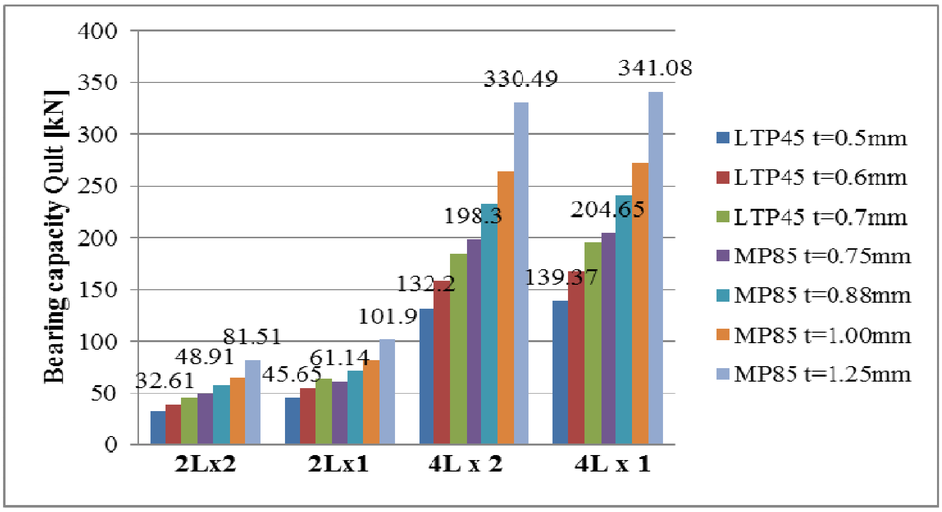


Figure 12: Loadbearing capacity for the roof diaphragm

2L x 2 – two sides fastening in alternate troughs (loadbearing capacity is decided by the sheeting/purlins fasteners);
2L x 1 – two sides fastening in every troughs (loadbearing capacity is decided by the sheeting/purlins fasteners);
4L x 2 – four sides fastening in alternate troughs (load bearing capacity is decided by the seam fasteners capacity) ;
4L x 1– four sides fastening in every trough (load bearing capacity is decided by the seam fasteners capacity).

5.2 Flexibility variation of the shear panels depending on the sheeting thickness

To account for a certain stiffness of the diaphragm panels in the 3D models' analysis, the total flexibility and stiffness of the panels were calculated according to:

- sheeting deformation;
- fasteners deformation;
- axial deformation of the purlins.

It can be noticed that depending on the fastening method, the flexibility of the roof shear panels are significantly smaller for every trough fastening (Cases 1, 3 – Figure 13) compared to alternate troughs fastening (Cases 2, 4 – Figure 13). As sheet thickness increases, the diaphragm stiffness also increases. The difference between the two sides and four sides fastening is represented by the flexibility of sheeting – rafter (through shear connectors), leading to a maximum 10% variation in results for sheeting thickness of less than 1 mm. For thicknesses equal to 1 and 1.25 mm, the flexibility is influenced by the profile distortion of the sheeting, leading to higher differences in flexibilities of the panels fastened in alternate trough (approximately 50 % - see Figure 13).

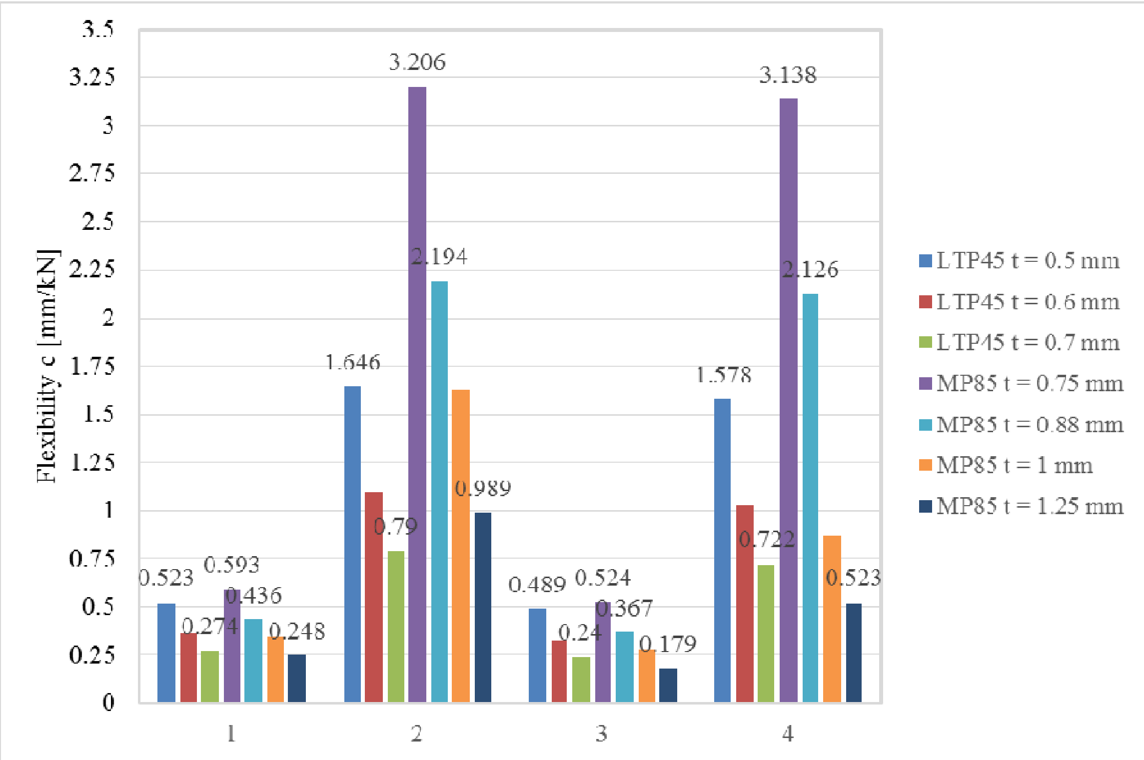


Figure 13: Flexibility variation of the shear panels depending on the fastening type

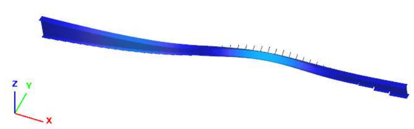
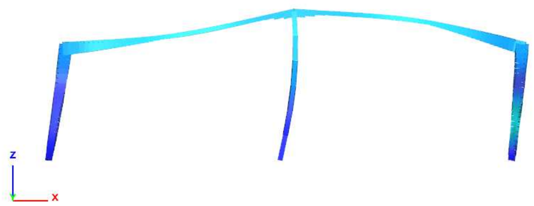
1 – two sides fastening, in every trough;
2 – two sides fastening, in alternate troughs;
3 – four sides fastening, in every trough;
4 – four sides fastening, in alternate troughs.

5.3 Variation of the critical load amplification factor α_{cr}

For different roof diaphragm models, the critical load amplification factor was computed, monitoring rafter buckling, column buckling and sway type frame buckling. The results are presented in Table 4 and in Figure 14, where α_{cr} - represents the ratio between the critical load and total load applied on the structure. The relevant combination used to compute the values of the load amplification factor is the one with permanent and snow loads.

Load amplification factor expresses the buckling resistance of an element as a component or of the structure as a whole. As the factor increases, the structure is more resistant to buckling. Therefore, it can be noticed that in the case of gross model – MB, the α_{cr} value is smaller because the frame is configured using only the main structural elements. If the MB model is improved with all the purlins and side rails and then with additional supports for stabilizing the cladding elements, transition models MT are obtained (without the sheeting); in this way we obtain a gradually increasing value α_{cr} for frame rafter. As the rafter is more stable, the frame column buckling will occur later, α_{cr} increasing from 2.27 up to 2.49. Until the sheeting effect is accounted for in the model (MB, MT1, MT2, MT3 model), there is almost no influence over the sway type buckling of the frame. When the roof sheeting effect is introduced in the model, an important increase in α_{cr} is obtained compared to the gross model MB. The values obtained in terms of buckling resistance in case of equivalent models ME, in which the shear panels are simulated, are even higher, but do not vary among the different equivalent models considered. MT3 can be considered a special case in terms of α_{cr} values for rafter. The higher value of the amplification factor is due to the use of link type elements with different axial stiffness values: in case of MT3 the links were considered with infinite axial stiffness, whereas in the equivalent model the calculated axial stiffness was imposed, following the procedure described in [6].

Table 4: Critical load amplification factor (α_{cr}) and critical load (P_{cr}) – computed with Consteel software

Rafter buckling			Column buckling	Frame sway type buckling	
					
Model/Element		α_{cr}	α_{cr}	α_{cr}	P_{cr} **[kN]**
MB		3.85	2.27	8.94	3382.22
MT 1		5.70	2.40	8.70	3291.42
MT 2		5.60	2.40	8.71	3295.20
MT 3		8.95	2.49	8.95	3386.00
2L	ME 1...7	6.17	4.44	14.90	5637.03
4L	ME 1...7	6.19	4.44	14.92	5644.60

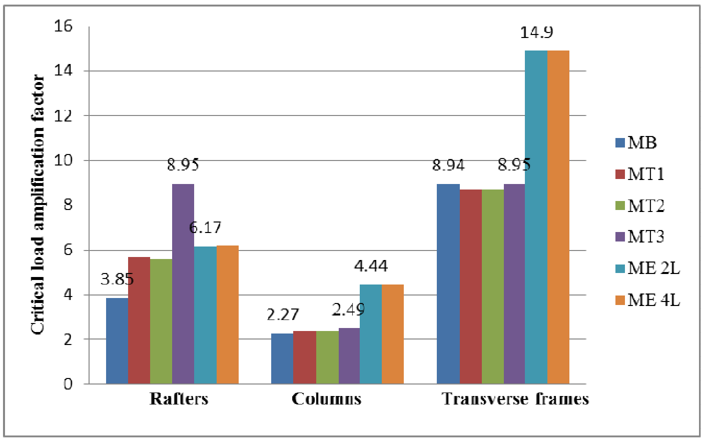


Figure 14: Computed critical load amplification factors

It is interesting to mention that the value of α_{cr} among the equivalent models does not depend on the trapezoidal sheeting profile or thickness or supporting method (two or four sides).

In terms of column stability, the load amplification factor increases gradually from the values obtained for the gross model towards the equivalent models values.

As far as the buckling mode is concerned, the rafter presents a lateral torsional buckling, while the column buckling mode is a torsional flexural one.

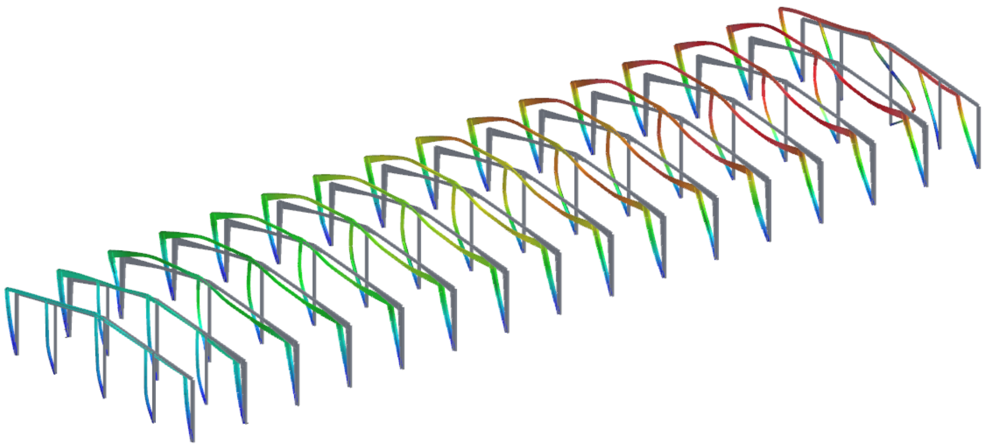


Figure 15: Sway buckling mode for the transverse frames

Sway buckling mode occurs at the same time for most of the transverse frames, according to Figure 15, the intermediate floor effect making more stable the neighbor frames.

In order to emphasize this aspect, a buckling sensitivity analysis was performed, obtaining results in terms of the most relevant buckling eigen shapes for columns. Figures 16 and 17 present the difference in terms of α_{cr} between the gross MB model, where the diaphragm effect is not applied and the equivalent ME model. It can be noted that the rafter has a higher buckling resistance compared to the column in the MB modeling stage. This aspect leads to a specific interaction between the two elements, illustrated by the load amplification factor α_{cr}.

In the ME model due to the stabilizing effect of the diaphragm it can be seen that there is no relevant buckling mode for the rafters' end, which also increases the α_{cr} value of the columns.

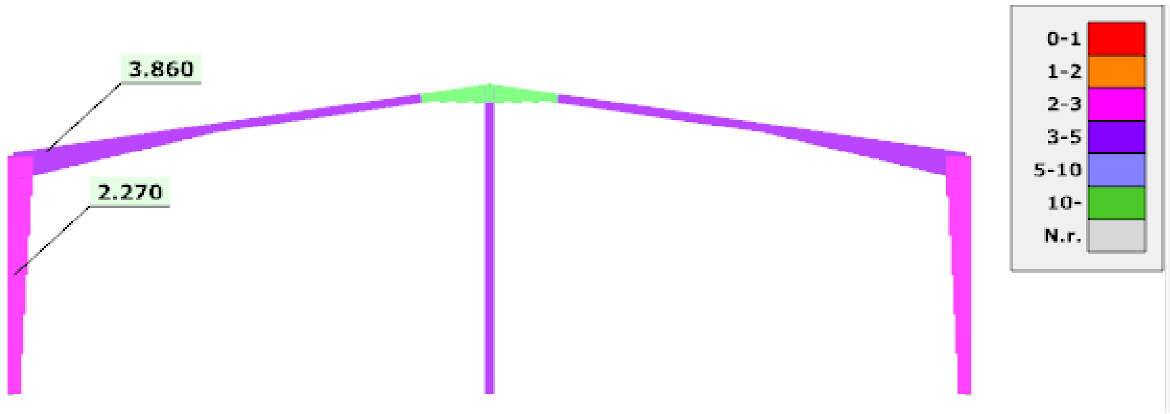


Figure 16: MB transverse frame – column α_{cr} vs. rafter α_{cr}

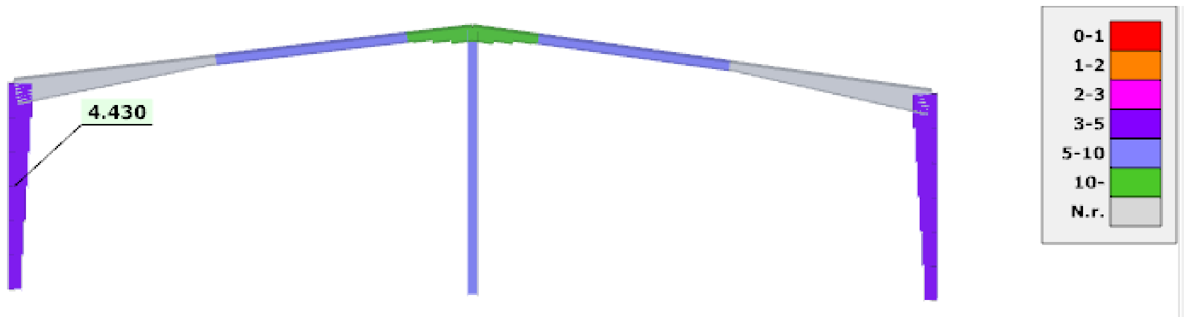


Figure 17: ME transverse frame – column α_{cr} vs. rafter α_{cr}

Considering the transverse frame as a whole, the α_{cr} values for sway mode buckling increase from the gross model to the equivalent models. We can observe almost no difference between basic MB and transition MT models, but an important increase can be observed if the stiffness of the roof diaphragm is computed. In this particular case, there is almost no difference in α_{cr} between two sides and four sides diaphragm, even though important differences in stiffness between equivalent models can be observed, as shown in chapter 5.2. The average α_{cr} values are presented in the chart (ME 2L, ME 4L) of Figure 16 and 17.

6 VALIDATION OF ANALYSIS RESULTS USING FEM

6.1 Preliminary numerical analyses

To confirm the analytically evaluated stiffening effect of trapezoidal sheeting, 3D FE model of the panel (12.40x6.00m) was developed using ABAQUS. The analysis type conducted is a nonlinear quasi-static displacement-based simulation using explicit dynamics. The model consists of shell elements considering just the Z purlins and the corrugated sheeting. The effect of the support of the purlins by a beam or frame was considered using appropriate boundary conditions. For the modelling of self-drilling screw connections between purlins and sheeting, CONN3D2 connector type has been used. The latter was set up to provide a semi-rigid behaviour similarly to self-drilling screws. In order to capture developments of local instability phenomena in the corrugated sheeting, the nonlinearity of the geometry was specified. The displacement is applied to a reference point which in turn is coupled at the end of the first Z-purlin. Similarly, the last purlin is attached to a reference point, restraining all degrees of freedom. In order to predict the shear stiffness of the model in transverse and longitudinal direction, two different load directions were considered.

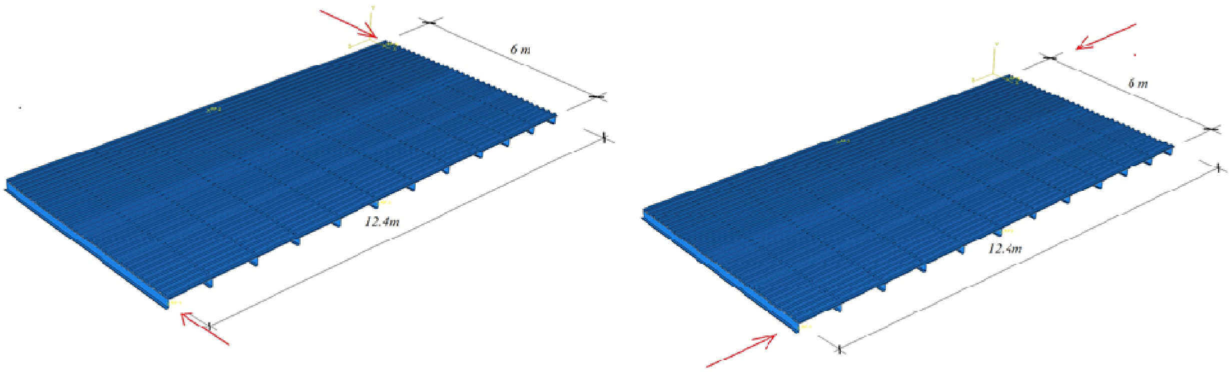


Figure 18: Geometry and shear loading of the model (transverse/longitudinal)

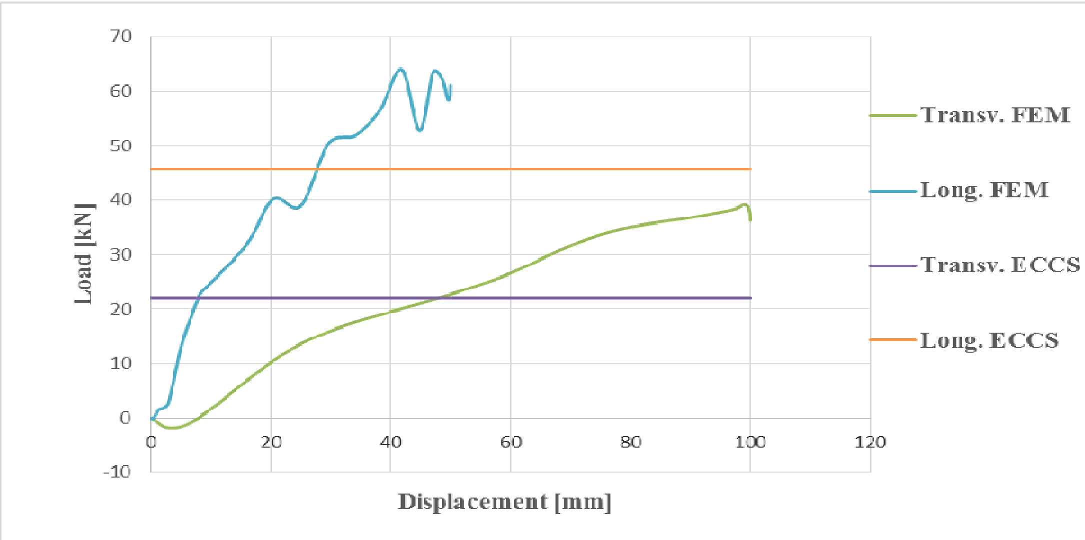


Figure 19: Load – displacement diagram for the transverse and longitudinal model

According to load-deflection curves computed using FEM model (Figure 19), the stiffness for loading parallel with the sheeting (longitudinal) resulted 2266N/mm (Figure 18 – right). The flexibility of the shear panel evaluated analytically according to [6] and [7] for one bay diaphragm was 0.47mm/kN. This value of the flexibility converted to stiffness has a value of 2127 N/mm, which is in good agreement with the FEM result. In case of loading perpendicular to the sheeting (transverse loaded shear panel) the resulted stiffness is 507 N/mm (Figure 18 – left). The values obtained in terms of the panel's shear capacity according to the manual calculations are satisfactory compared to the values given by nonlinear analysis computed in Abaqus (see Figure 19).

As preliminary evaluation of the load multiplication factor using Abaqus, a 2D frame model was developed. The model consists of shell elements and Subspace Eigensolver is used for the buckling analysis. The model has fixed supports on longitudinal direction and is loaded with a unit force equal to 1N which is applied to a reference point. The reference point is coupled to the upper chord of the beams using structural distributing coupling method, as presented in Figure 20. This loading model is equivalent with uniform distribution of the unit load on the upper flange of the beams. The eigenvalues thus obtained corresponds to α_{cr} and in this case this is equal to the critical load.

Figure 20: FE model for 2D frame with fixed lateral supports

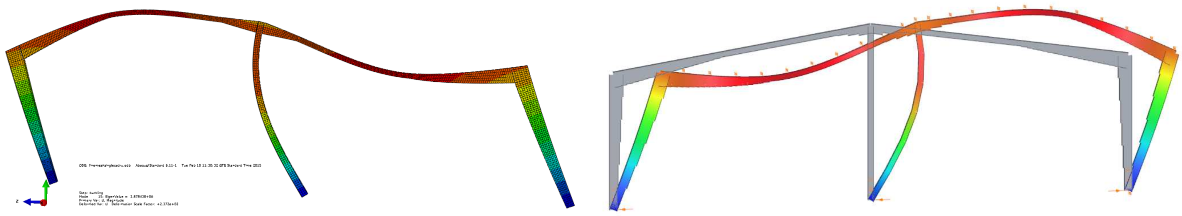


Figure 21: Sway buckling for a 2D frame with fixed supports on longitudinal direction – Abaqus(left), Consteel (right)

Table 5: Critical load for a 2D frame

Model	**Frame sway type buckling Critical load [kN]**
Consteel	3302
Abaqus	3787

A buckling analysis on a similar 2D frame model was also computed with Consteel software. The difference between the two computed critical load values is around 15%. Further studies (Figure 22) are under development to evaluate the α_{cr} value in ABAQUS using buckling analysis, while considering a FEM model with two frames and the related roof sheeting. This model will be used as a benchmark, to compare the results with the simplified ConSteel bar model. Due to model complexity and analysis time, results are not yet available.

Figure 22: Complex frame and roof model and preliminary sway buckling computed in Abaqus

7 CONCLUSIONS

The paper focuses on a structural problem with complex issues, formulating design procedures developed for a reduced range of analysis models. The objectives of this study are based on the assumption that in the case of a steel structure, with pitched roof portal frames, which has corrugated sheeting as cladding, there is a structural strength reserve due to the diaphragm effect of the sheeting which increases the stability of the members (rafter and column) obtaining a higher load carrying capacity for the whole frame. This capacity reserve exists, even if it is not accounted for in the design process. The idea was to quantify these reserves using the existent design procedures ([6], [7]) by adding a simplified analysis method to the analytical computations.

After processing the results obtained through manual calculations and design models, the following conclusions can be drawn:

1. From the point of view of load bearing capacity, the shear resistance of the roof panels depends on the seam fasteners capacity, type and thickness of the sheeting and side fixing. The shear capacity can increase four times from two sided fastenings to four sided fastenings, as shown in Figure 12.
2. In terms of shear capacity of the seam fasteners (F_s) and of the sheeting/purlin fasteners (F_p), using the design formula recommended by the Romanian code [7] leads to quite conservative results. Due to this reason ECCS recommendations [6] and Davies and Bryan values [2] were used, excluding the values given by the Romanian code. Authors recommend a revision of the expressions mentioned above in reference [7], which give half of the F_s and F_p shear capacity values.
3. The flexibility of roof shear panels increases with 60-90% if the sheeting is fastened only in alternate troughs compared to every trough, in both cases of support type (two sided or four sided), as shown in Figure 13;
4. By applying the diaphragm effect on the designed structures, the critical load amplification factor α_{cr} increases significantly from the gross model MB to the equivalent models ME (Figure 14);
5. Even the computed critical load is a theoretical one which cannot be achieved in reality, taking into account the stressed skin action, there is an important increase in the critical load of the frame, which in this particular case represents an aproximately 50% increase compared with the computed critical loads for simplified models, where diaphragm action was not present;

6. Developed equivalent models seem to be insensitive to stiffness variation of the analyzed roof diaphragms, the computed critical load resulting in all of the cases more or less the same.

FE models of the shear panels and 2D frame structure were created in Abaqus. The values obtained in terms of flexibility of the shear panel are in good agreemnent with the manual calculation. In case of the 2D frame, similar buckling eigen shapes and critical load values were obtained, both in Abaqus and in ConSteel software. Due to the particularity of the case study, conclusions should be relevant only for the analysed structure. To be able to state general conclusions, the research continues with FEM modelling, to validate the results for other structural configurations and different geometries.

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