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NON-LINEARITY OF CONNECTIONS IN THE EQUIVALENT CANTILEVER BEAM METHOD (ECBM) FOR STRESSED SKIN DIAPHRAGMS MODELLING

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Abstract: Stressed-skin diaphragm action occurs when cladding, typically thin steel sheets or two sheets encapsulating insulation, significantly contributes to a structure's global stiffness. Ignoring this effect in design can lead to inaccurate assessment, particularly under horizontal loads. The Equivalent Cantilever Beam Method (ECBM), considered an extension of the Component Method, models the flexibility of connections using equivalent cantilever beams composed of standard finite element members available in engineering analysis software. Earlier applications of the ECBM were limited to the elastic range of connector behaviour. This study introduces a nonlinear extension of the method by incorporating material yielding into the equivalent beams, enabling plastic redistribution of forces without specialised nonlinear springs. A hierarchical validation is presented: single-connector behaviour, a diaphragm panel against tests and ECCS predictions, and a full 3D building against an advanced reference model. Results demonstrate excellent agreement, indicating that the proposed method provides a practical and deployable solution for including stressed-skin effects in routine structural design models. The ECBM still follows the analytical framework of ECCS Recommendations No. 88, extending its applicability while maintaining its original simplifications and assumptions. Despite these limitations, the method may support the practical assessment of clad steel structures, including cases involving parasitic diaphragm action and the possible reduction or elimination of roof-plane bracing.

Key words: structure cladding interaction, stressed skin theory, diaphragm action, component method

1. INTRODUCTION

1.1. Stiffening effect of steel cladding

The stiffening effect of steel cladding was first recognized in the late 1950's [1] when it was found that the measured deflections of a clad structure were significantly lower than predicted. It soon became clear that the assumption that intermediate frames resist the entirety of applied actions was wrong; instead, the cladding system, which has high in-plane stiffness, was spanning between the (stiff) clad gable frames and some proportion of the applied actions were resisted via this alternative load path as shown in Fig. 1.

Further tests were conducted, and a basic theory was presented by Bryan [2]. This made use of a simple equilibrium model to predict the strength and stiffness of a single "roof panel" (defined as the area between adjacent frames on a roof slope), and a compatibility model which interfaced the panels with the frames. The equilibrium model was refined by Davies [3], who used simple Finite Element Models (FEMs) of cantilever shear panels to validate the equations. At this time due to computational limitations only a relatively modest panel could be modelled with FEM, and for design the derived equations together with the compatibility model were the only option. The theory was published in 1982 by Davies and Bryan [4], which, with some additional important contributions, was published in the British Standards in 1994 [5] and the European design guidance in 1995 [6]. EN-1993-1-3 [7] contains enabling clauses for stressed skin design, and references ECCS for the detailed design procedure making it a current document despite being published almost 30 years ago, and predominantly based on research conducted over 40 years ago.

The design methodology contained within ECCS [6] has never, to the best of the authors knowledge, been applied to pitched roof portal structures; instead, the cladding is invariably ignored during design and assumed to act as a system to resolve forces back to secondary steel work (purlins or rails). Experiments, and validated FEMs by the authors show that under horizontal actions the forces in design models are entirely fictitious [8]. The issue is exacerbated as frames have become more flexible due to increases in scale [9] and the adoption of cold-formed steel for primary frames [10], while cladding systems have become more complex and often stiffer [11]. Damage to cladding components (panels, fasteners, purlins and cleats) has been observed in real structures due to parasitic diaphragm action [9], while laboratory tests suggest that primary frames constructed from cold-formed steel are susceptible to damage due to the built up shear forces [10]. The available code methods [6] have been found unsatisfactory to accurately predict such damage [12,13], and models are typically used which disregard the global stiffening effect of cladding and produce inaccurate results under horizontal loading. There therefore exists a need to define a code compliant, accurate and robust method of augmenting existing 3D analysis models to include the cladding. Any method must model the entire cladding system in sufficient detail to allow assessment of all potential failure modes in the cladding, the interface between the cladding and the structure (purlin/cleats), and the primary structure itself while providing a sufficiently accurate global response.

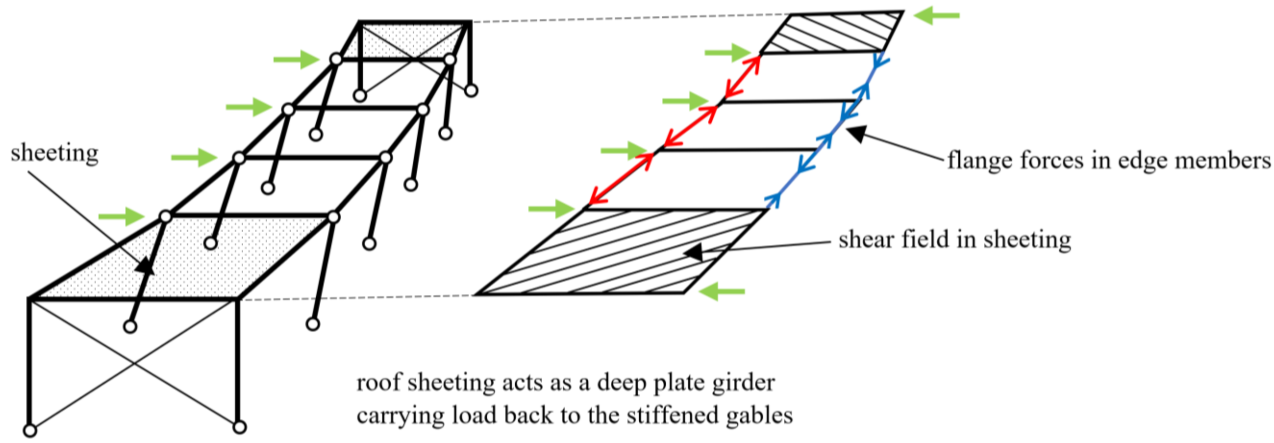

Fig. 1. Diaphragm action of a clad structure

1.2. Numerical modelling and simplifications for diaphragm analysis

When standard analytical procedures are not adequate for a particular design case, numerical modelling may be used to represent diaphragm action with different levels of detail. The simplest numerical idealisations reproduce the global diaphragm response by equivalent springs or members of calibrated flexibility [14] (also known as the ETM – Equivalent Truss Method [15]). Such models are useful for estimating the overall stiffness contribution of cladding to the primary frame, but they do not directly describe the distribution of forces between individual fasteners, seams, purlins and cleats. Instead, these local effects are represented only in an aggregated form, through the resultant diaphragm stiffness assigned to the equivalent members.

At the other end of the modelling spectrum, detailed component-based finite element models represent the main physical parts of the diaphragm explicitly, as demonstrated in one of the early works in this field [16]: fasteners are usually modelled by springs, purlins by beam elements, and sheeting by orthotropic shell, plate or plane-stress elements. These models can provide a high-fidelity representation of diaphragm behaviour, including nonlinear response and progressive force redistribution, but they often require specialist nonlinear elements, advanced modelling procedures, or software features that are not available in routine structural design packages.

More advanced and highly detailed approaches have also been used in studies devoted to the simulation and behaviour of single-span portal frames [17,18]. Comprehensive simulations of complete buildings were performed with the aim of reproducing the structural behaviour as realistically as possible, using models developed in SIMULIA Abaqus. Primary connections between cladding and secondary cold-formed steelwork, such as sandwich-panel-to-purlin connections, were modelled using beam elements representing long screws connecting the panel sheets, with several springs attached to reproduce the detailed behaviour of joint components. Sheet-to-sheet connections, i.e. seams, were treated as secondary connections and modelled using a special element type (*FASTENER) or typical multipoint constraints (MPCs) supplemented with additional springs. These approaches provide valuable research insight into the behaviour of complete portal-frame buildings and the mechanisms of cladding–structure interaction. At the same time, their use in routine design practice is naturally constrained by the high level of modelling detail, specialist software requirements and computational effort involved.

The Equivalent Cantilever Beam Method (ECBM) is positioned between the first two simplified approaches. It retains the component-level description of the cladding system where this is important for engineering interpretation, but replaces specialist nonlinear springs with equivalent beam elements that can be defined in standard finite element software. In this way, ECBM aims to provide a practical modelling route for routine 3D structural analysis while still allowing the local behaviour of connections to influence the global response of the building.

1.3. Equivalent Cantilever Beam Method (ECBM) - development background

The development of the Equivalent Cantilever Beam Method (ECBM) was motivated in part by the ECCS [6] statement that standard procedures are applicable mainly to single-bay flat roofs and symmetrical single-bay pitched roof frames, whereas more complex cases require computer analysis. In simplified numerical idealisations, diaphragm action may be represented by equivalent springs or members of a given flexibility c. Such flexibility may, for example, be represented as a member of length L, modulus of elasticity E and equivalent cross section area A [6]:

$$A = \frac{L^3}{cb^2E} \tag{1}$$

This type of formulation refers to the diaphragm response at a

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global level. By contrast, the ECBM developed in the present work operates at the level of individual connections, whose flexibility is reproduced by equivalent cantilever beam elements. Thus, although the mechanical parameters are different and apply to different structural components, both approaches share the broader modelling idea of reproducing structural flexibility through calibrated equivalent finite elements.

Moreover, representing diaphragms by single equivalent springs may be insufficient when a more detailed assessment is required, particularly with regard to the local behaviour of panels and connections. As noted in the previous section, practical design problems may require not only the estimation of the global stiffening effect, but also the evaluation of local force transfer and potential damage mechanisms. The Component Method of modelling cladding addresses this need by representing the main roof components explicitly using simple finite elements. Each roof component is explicitly modelled by a simple finite element. Historically, springs were used for fasteners, beam elements for purlins and orthotropic tri/quad elements for the sheeting. Recently these cladding models, which have been derived for most common cladding systems in use today, have been validated against cantilever tests [19] and demonstrate that it is possible to predict the full nonlinear panel response all the way to failure using the Component Method. These validated models have also been interfaced with the 3D building analysis models typically used for design and validated against test results [20]. One of the drawbacks of the component method is that it requires specialist elements (yielding springs) which are not generally available in commercial analysis and design software. The Equivalent Cantilever Beam Method (ECBM) has been developed to address this potential limitation. The main aim was to retain all the advantages of the Component Method, which provides an intuitive and robust method of structural modelling of industrial buildings covered by metal sheeting, but to model fasteners using elements commonly available in less specialised analysis and design software.

The ECBM was presented by the first author in 2015 [19]. A simple roof panel acting as a diaphragm was modelled using the Finite Element Method (FEM). The results were compared with laboratory tests and provided a good estimate of the elastic stiffness of the panel. Next, a complete building structure was tested in three phases: with no cladding, with just the roof clad and finally considering walls sheeting. The structure was initially modelled with no cladding, which matched well with the experimental results from phase one. Roof cladding was then added using the ECBM, which corresponded to phase two and finally to the walls which corresponded to phase three. In all three phases a good agreement with the linear response of the structure was found, validating the ECBM [21]. The same building was later used as validation for the Component Method, and a full nonlinear analysis of the structure performed to failure under horizontal load [8].

Since then, the focus has been on the problem of using the available commercial structural analysis and design software to simulate the behaviour of buildings with attached sheeting. Such software is already owned by most consultants, and including the cladding through the ECBM is a natural extension of a design model, rather than a separate design method [6] or one requiring bespoke software to perform advanced numerical analyses [22]. The main principle of the method is to use simple beam elements to simulate complex connections in models of cladding systems. This approach allows diaphragm modelling using software not specifically prepared for this purpose. The basic literature, theory, numerical, analytical and laboratory experiments were summarised in the first author's PhD Thesis "The method of modelling of steel halls structures covered by trapezoidal sheeting" (in Polish) [23]. It is worth mentioning that, to date, the comparable methods or their basic theoretical concepts have already been independently used as a common solution [24,25] or referenced to the author's works by other researchers in their publications [26–28].

One of the common problems in diaphragm analysis is that there can be a significant number of small fasteners yielding and even failing at relatively low load levels. Individual fastener yield/failure does not necessarily constitute a failure of a roof panel, which often shows nonlinearity at low load levels but cannot be said to have failed until a recognized failure mode has occurred. This obviously necessitates a nonlinear analysis which accurately predicts the failure of fasteners, and allows forces to redistribute through the panel. The previous versions of the ECBM were sufficient to analyse simple diaphragms considering their initial stiffness stage which could match the analytical assumptions in the ECCS [6,29]. These, however, could not be used to predict failure modes or the ultimate limit state of a panel. It was thus deemed essential to enhance the method to allow more advanced simulations, moving further than conservative theory, and allowing better recognition of the failure of a structure. The cooperation with Roberts & Davies [8], who developed their own advanced non-linear models, brought new insight into the idea and was the inspiration to enhance the ECBM. As a result of this research on building analysis with diaphragms, a new set of data has been prepared. This data has been used in this paper as the first step of the method validation, using the non-linear characteristics of connections. Furthermore, for the need for this validation, other results of detailed laboratory research on connections have been used to prove that the proposed solution satisfies the behaviour of real joints.

2. THE EQUIVALENT CANTILEVER BEAM METHOD (ECBM)

2.1. Basis of the diaphragms modelling methodology using ECBM

The ECBM is based on the Finite Element Method, providing the possibility to add diaphragms to the framed structures numerical models. The procedure could be started with the usage of European Recommendations [6] to obtain the necessary properties of the diaphragm's components. The sheeting can be simulated as an orthotropic continuum (e.g. plate or plane-stress elements). The main problem, that the ECBM solves, is the interface between structural elements (plates to plates and plates to beams). These elements can be connected using ECBs (equivalent cantilever beams) in the way that allows the correct relationship with connected elements, leading to the successful convergence of calculations. Thus, special attention should be placed on the support conditions of an ECB (Fig. 2).

One end of an ECB is "free" and can be connected with another element by releasing the rotational degrees of freedom if this element has a bending stiffness defined (which is common for typically used solids, plate elements or beams). Another end has to be "rigid" and is more challenging in application. When the connected bearing element has high bending stiffness compared to the assumed ECB (e.g. beam or purlin), then the problem is trivial and can be verified, e.g. by unit simulation of a single ECB if necessary (joints with beams usually fulfil this requirement). For situations when the connected element formulation does not provide the bending stiffness or is particularly considerably low (e.g. thin sheets), additional support conditions are necessary to be defined at this point (fixed rotations).

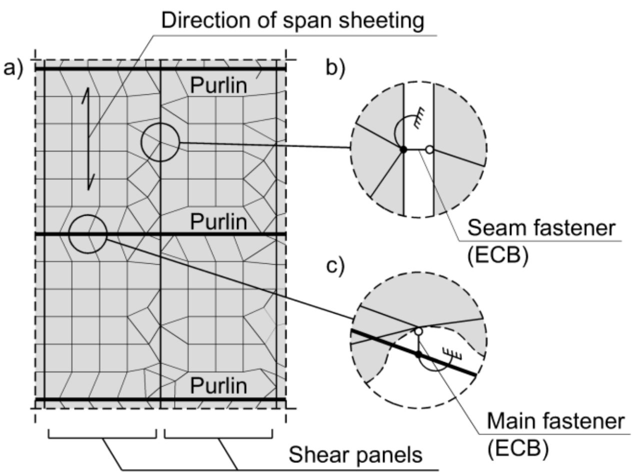

Fig. 2. Diaphragm modelling using ECBs: a) typical model of a diaphragm; b) seam fastener viewed from above; c) vertical main fastener in 3D

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2.2. Assumptions and parameters selection

In general, the behaviour of connections (Fig. 3) can be described as their flexibility (or stiffness as the inverse of flexibility) connected with their slip. In the ECBM a connector is represented by an equivalent cantilever beam (ECB). The calibration target is the connector's elastic flexibility s (target values can be taken from tests or literature [4,6]). Given a chosen Young's modulus E and equivalent length L_{eq}, the equivalent moment of area follows from the cantilever end-deflection formula (compare with Fig. 3):

$$s = \frac{L_{eq}^3}{3EI_{eq}} \tag{2}$$

Thus, equivalent moment of inertia can be obtained as:

$$I_{eq} = \frac{L_{eq}^3}{3sE} \tag{3}$$

It should be clearly noted that, in the ECBM, the equivalent cantilevers represent the resultant force–displacement behaviour of the complete connection assembly, including the combined effects of sheet bearing, fastener tilting and other local mechanisms. These mechanisms are therefore incorporated in an equivalent, aggregated manner rather than modelled as separate physical failure modes. The quantities L_{eq} and E do not represent physical material properties of the fastener. They are modelling parameters, chosen to:

- avoid node-merging and minimum-length tolerances in the FE solver, or
- maintain a structural distance between the centre of gravity plane of connected sheets, if necessary, and
- provide a well-conditioned stiffness assembly.

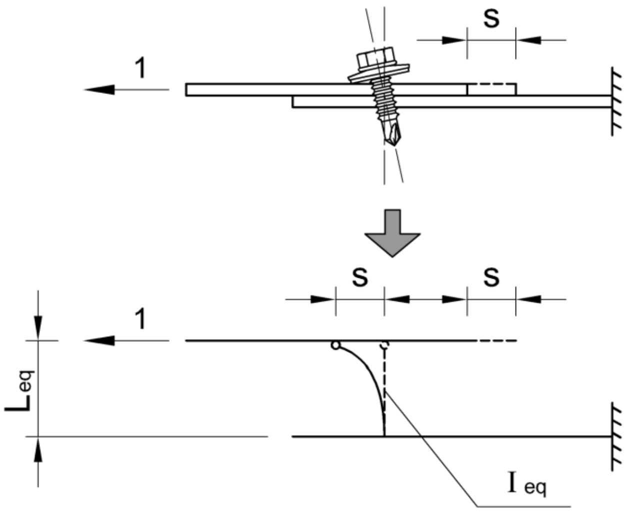

Fig. 3. The main concept of the equivalent cantilever beam method (ECBM)

2.3. Non-linearity of connections in the ECBM as a solution for connections resistance

Until now, usage of the ECBM for all simulations was carried out only in the elastic range with the main focus on obtaining a more realistic structural behaviour without paying special attention to the strength of components. Recent research by Roberts, Davies and Wang [30,31] demonstrated that simulations considering the plastic behaviour of components with their proper capacity assessment are crucial. When the ECBM was first presented [23], introduction of nonlinearity was one of the identified tasks necessary to improve the developed approach.

The first proposition (**Fig. 4**) was to modify the calculated value of the equivalent moment of inertia I_{eq} depending on the slip of a connection calculated under a load at a specific load step (concept I). This approach would demand the definition of a function, where the variable will be a moment of inertia. Another idea was to add a spring element (or an additional equivalent beam) responsible for the non-linear stage (concept II). The two propositions appear to be complicated or impractical to implement. The first concept requires checking or updating a geometrical property of a cantilever beam at every step of non-linear calculations. This is not a standard feature of typical FEA software but is possible in advanced ones. The second concept is to use another medium, a spring or a beam element, being responsible for the next order stages of the non-linear work of a connection. Again, such an approach would be possible if the software would allow e.g. non-linear springs which could work beyond the elastic range. Nevertheless, combining a cantilever beam in an elastic range with extra inelastic elements is a redundant action since both stages could be realized using a single non-linear component for the complete spectrum.

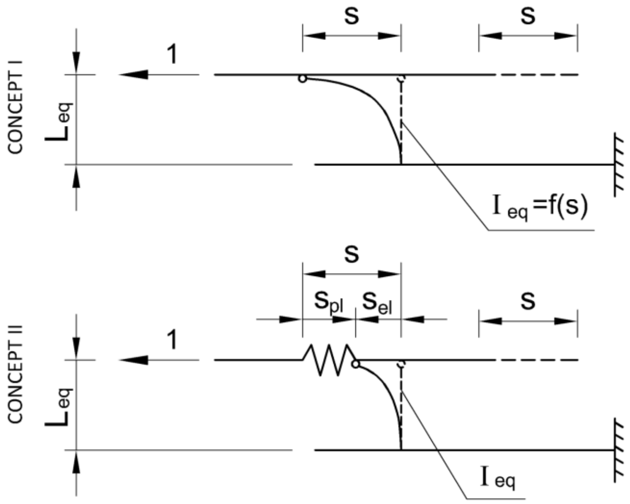

Fig. 4. Two past out-of-date propositions of the extension of the ECBM by a non-linear stage [23]

The current idea is to take advantage of the simplicity of the ECBM, consequently maintaining the original concept of the equivalent cantilever beam in the scope of the geometry, and modifying its material properties only to introduce nonlinearity of connections. This will allow the inclusion of a controlled yielding of these equivalent cantilevers being representatives of each connection in a system. Until now, the only input required for the calculations was the value of this property. Now, to incorporate non-linearity, it is necessary to choose a particular cross-section and evaluate the equivalent moment of inertia I_{eq} in relation to it. Between countless possibilities of potential cross-sections, two types can be introduced as an initial input to the basic theory: namely circular and rectangular.

The solid circular cross-section is suitable for typical connections when it is possible to assume uniform flexibility in all directions. E.g. those are main connections (sheet to bearing element) and seams (sheet to sheet) in diaphragms. The moment of inertia of the equivalent circular cross-section is:

$$I_{c,eq} = \frac{\pi d_{eq}^4}{64} \tag{4}$$

where d_{eq} is the diameter as the only input necessary to define the cross-section. Considering that $I_{c,eq} = I_{eq}$:

$$d_{eq} = \sqrt[4]{\frac{64 I_{eq}}{\pi}} \tag{5}$$

To incorporate stress analysis (to assess yielding) it is necessary to specify plastic section modulus:

$$W_{pl,eq} = \frac{d_{eq}^3}{6} \tag{6}$$

An equivalent bending moment at the base of the cantilever beam model corresponds to a shear capacity of a connection F_r:

$$M_r = L_{eq} F_r \tag{7}$$

The bending resistance of a beam is connected with equivalent plastic (critical) stress $f_{eq,}$ and plastic section modulus $W_{pl,eq}$, then:

$$M_r = W_{pl,eq} f_{eq} \tag{8}$$

Finally, the cantilever material equivalent critical stress f_{eq} is:

$$f_{eq} = \frac{6L_{eq}}{d_{eq}^3} F_r \tag{9}$$

If a load direction is important, then the rectangular cross-section can be used to take into account different values of connection flexibilities. This can be useful e.g. for modelling purlin cleats which usually have different properties in the two directions (e.g. s_x and s_y). Thus, the equivalent moment of inertia has to be divided into:

$$I_{x,eq} = \frac{L_{eq}^3}{3 s_x E} \tag{10}$$

$$I_{y,eq} = \frac{L_{eq}^3}{3 s_y E} \tag{11}$$

A relationship between the two moments of inertia p can be written as:

$$p = \frac{I_{y,eq}}{I_{x,eq}} \tag{12}$$

such that:

$$I_{y,eq} = p I_{x,eq} \tag{13}$$

If h_{eq} is the height of the rectangular cross-section (parallel to the vertical y axis), and b_{eq} is its width, then:

$$\frac{h_{eq} b_{eq}^3}{12} = p \frac{b_{eq} h_{eq}^3}{12} \tag{14}$$

$$b_{eq}^2 = p h_{eq}^2 \tag{15}$$

$$b_{eq} = h_{eq} \sqrt{p} \tag{16}$$

Following the above considerations, the main relative equivalent moment of inertia can have the form:

$$I_{x,eq} = \frac{h_{eq}^4 \sqrt{p}}{12} \tag{17}$$

Finally, necessary dimensions of the equivalent rectangular cross-section can be obtained as follows:

$$h_{eq} = \sqrt[4]{\frac{12 I_{x,eq}}{\sqrt{p}}} \tag{18}$$

$$b_{eq} = h_{eq} \sqrt{p} \tag{19}$$

Stress analysis, in this case, is not as straightforward as in the circular section. The strength can be potentially different in both directions, not correlating with the uniform geometry of the equivalent rectangular section. The possible solution could be maintaining proportions between section moduli in both directions similar to both strengths, respectively. However, the initial concept is to choose a possible lowest strength and estimate the equivalent yielding (critical) limit stress for a connection. It will be a simplified but safe assumption until a more appropriate approach is developed. Then, if:

$$W_{pl,x,eq} = \frac{b_{eq} h_{eq}^2}{4}, \; W_{pl,y,eq} = \frac{h_{eq} b_{eq}^2}{4} \tag{20}$$

$$M_{r,x} = L_{eq} F_{r,x}, \; M_{r,y} = L_{eq} F_{r,y} \tag{21}$$

$$f_{x,eq} = \frac{M_{r,x}}{W_{pl,x,eq}} = \frac{L_{eq} F_{r,x}}{\left(\frac{b_{eq} h_{eq}^2}{4}\right)} = \frac{4 L_{eq} F_{r,x}}{b_{eq} h_{eq}^2} \tag{22}$$

$$f_{y,eq} = \frac{M_{r,y}}{W_{pl,y,eq}} = \frac{L_{eq}F_{r,y}}{\left(\frac{h_{eq}b_{eq}^2}{4}\right)} = \frac{4L_{eq}F_{r,y}}{h_{eq}b_{eq}^2} \tag{23}$$

the critical stress assumed for the connection can be:

$$f_{eq} = \min\left(f_{x,eq}; f_{y,eq}\right) \tag{24}$$

$$f_{eq} = \min\left(\frac{4L_{eq}F_{r,x}}{b_{eq}h_{eq}^2}; \frac{4L_{eq}F_{r,y}}{h_{eq}b_{eq}^2}\right) \tag{25}$$

For a detailed analysis, an exact type of cross-section can be chosen considering the shape factor [32] which is defined as the ratio of the plastic moment to the yield moment. This can help to improve the ECB non-linear behaviour allowing matching with the assumed connection force-deflection curve. The solid circular section has a 1,67 value of shape factor. The rectangle or square has 1,5. For comparison, the I-section can have 1,13. The lower the value of the shape factor, the faster the plastic moment is reached by the element following initial yield. In conclusion, the shape factor allows for the calibration of ECB behaviour. These alternative behaviours are demonstrated in Fig. 5, which shows shapes configured to have the same elastic stiffness and plastic bending resistance. However, the choice of shape affects the onset and rate of nonlinearity. This topic will be explored in more detail in a future paper.

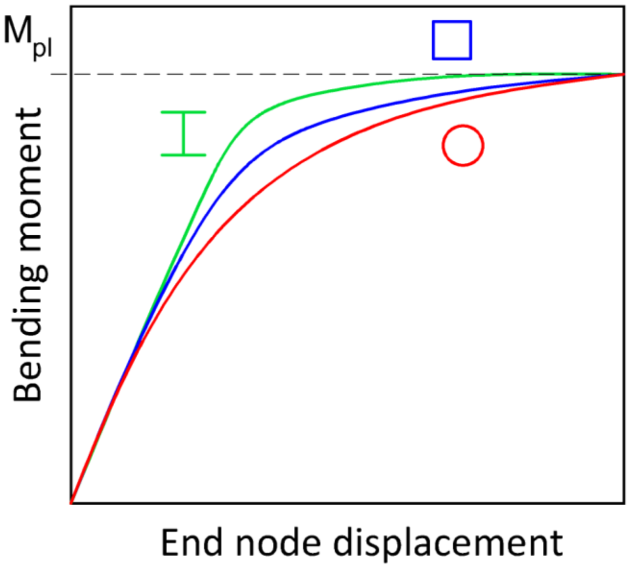

Fig. 5. Schematic influence of the shape factor on the behaviour of a cantilever

2.4. The problem of modelling the purlin to rafter connections (cleats)

The considerations from the previous section relate to connections between purlins and rafters. Historically, the only known data available for the stiffness and strength of purlin cleats was based on tests conducted in the 1960's [33] which have subsequently been reproduced in various publications [4,6]. However, these cleats are distinct from those used today.

Recent publications by Roberts & Davies [11,12,31] proved that current research into the performance of purlin cleats needs to be updated, e.g. in the field of their flexibility of the strength and flexibility in both directions, under static load and (which can be crucial) under cyclic loading. The case is that the modern bearing structures (connections types also) were changed in the scope of their geometries regarding to the classic theory ("effect of scale"). The stressed skin effect can no longer be positive, it indeed can be tragic in its consequences, leading to the destruction of the structure in an unexpected mode (unwanted "parasitic diaphragm action"). Thus, the classic solutions applied e.g. to large warehouses ("big sheds") appear to be not sufficient to properly assess the structure strength. The following text directly addresses this problem [8]:

"However, lying behind the problem of design values for purlin cleat performance is a fundamental issue. In recent years, a Big Shed has been declared to be unsafe and been closed for complete re-roofing as a consequence of a progressive failure in the purlin cleats. Further investigation revealed that the cause was fatigue after more than 20,000 cycles of wind gust loading. Once the first cleat failure had taken place, further failures followed, and cleats were fracturing faster than they could be replaced. Regrettably, the details this building failure are not in the public domain. However, the fact remains that purlin cleat failure has always been a potential failure mode under (parasitic) stressed skin action and it is now abundantly clear that it cannot be ignored".

In conclusion, the role of purlin cleats has been highlighted as one of the key unresolved issues in this field. Nagy et al. [13] validated a cleat model using the available experimental results [33] and proposed expressions for the stiffness of a modern "welded C" cleat, commonly used in Romania, recognising the different "upslope" and "down-slope" behaviour. More recent work has also presented a broad study on two-side fastened trapezoidal sheet diaphragms, with particular attention paid to diaphragm-to-rafter connections [34], further confirming the importance of purlin connections in diaphragm action. These studies show that the behaviour of purlin-cleat connections remains an active research topic. The authors would like to clarify that the structures used for validation in the present paper were not governed by purlin-cleat nonlinearity or cleat failure. Therefore, for comparison with the reference analyses and within the practical scope of the present study, the cleat behaviour was modelled according to the available classical stiffness assumptions. This simplification was considered acceptable for the validation cases analysed here, since their response was not governed by purlin-cleat nonlinearity, sign-dependent stiffness or cleat failure. Nevertheless, future developments of the ECBM should include appropriate techniques for representing purlin-cleat connections with a more realistic, and possibly nonlinear, behaviour.

3. UNIT DIAPHRAGM AND A SINGLE ECB ANALYSIS (BASIC VALIDATIONS)

The basic validation analysis is carried out based on past research [19,23] in which a single shear diaphragm being part of a roof was calculated in the linear spectrum. The diaphragm dimensions in the outer beams axes are 4,56 m by 2,34 m (Fig. 6).

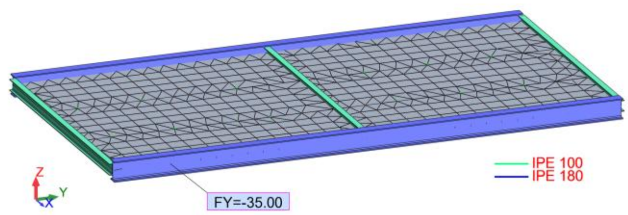

Fig. 6. Unit diaphragm model view (dimensions in the outer beams axes: 4,56 m and 2,34 m)

One rafter (fabricated of IPE180) is fixed, whereas the opposite is loaded up to a load of 35 kN. Now, the numerical model is rebuilt using non-linear properties of connections and compared to the experimental results. Additionally, to improve the connection behaviour, a single ECB is compared to the design model obtained from another laboratory test [35].

3.1. Properties of Equivalent Cantilever Beams (ECBs)

The properties of connections (Tab. 1) are calculated using formulae presented in the second chapter of this paper. Unlike the sheet fasteners, the cleat behaviour is considered to be linear (more research is still required in this field).

3.2. The single ECB analysis results

The work of seam fasteners (sheet to sheet) has been checked by modelling a single connector (Fig. 7). The resultant deflection curve of the free node of cantilever is compared to the connection model built based the tests on similar connections of the two thin plates [35]. The connection model is for the 0,60 mm thickness of both plates made of DX51 steel, connected with a HILTI S-MD screw similar to those used in this case.

Tab. 1. Properties of Equivalent Cantilever Beams for unit diaphragm (ECBs)

Variable description	Input and output values			
	Symbol	Unit	Value (CHAPTER 3)	Value (CHAPTER 4)
Seam fasteners (sheet – sheet)				
Fastener shear flexibility	s_s	mm/kN	0,25	0,25
Young Modulus	E	GPa	210,0	210,0
ECB length	L_{eq}	mm	2,0	2,0
Required moment of inertia	I_{eq}	mm^4	0,05079	0,05079
Cross-section type	circular			
Cross-section dimension (diameter)	d_{eq}	mm	1,0086	1,0086
Shear resistance	F_r	kN	1,20	1,20
Behaviour	non-linear			
Material equivalent yield (critical) strength	f_{eq}	MPa	14035	15204
Main fasteners (sheet – purlin)				
Fastener shear flexibility	s_s	mm/kN	0,15	0,35
Young Modulus	E	GPa	210,0	1,0
ECB length	L_{eq}	mm	2,0	1,0
Required moment of inertia	I_{eq}	mm^4	0,08466	0,95238
Cross-section type	circular			
Cross-section dimension (diameter)	d_{eq}	mm	1,14597	2,0987
Shear resistance	F_r	kN	2,63	2,50
Behaviour	non-linear			
Material equivalent critical stress	f_{eq}	MPa	21302	1620
Shear fasteners (sheet – shear connector) *				
Fastener shear flexibility	s_s	mm/kN	0,15	-
Young Modulus	E	GPa	210,0	-
ECB length	L_{eq}	mm	12,0	-
Required moment of inertia	I_{eq}	mm^4	18,28571	-
Cross-section type	circular			-
Cross-section dimension (diameter)	d_{eq}	mm	4,39325	-
Shear resistance	F_r	kN	2,63	-
Behaviour	non-linear			-
Material equivalent yield (critical) strength	f_{eq}	MPa	2222,95	-
Purlin cleats (purlin – rafter)				
Cleat shear flexibility – x-x	$s_{s,x}$	mm/kN	0,84	0,50
Cleat shear flexibility – y-y	$s_{s,y}$	mm/kN	0,005	0,0012
Young Modulus	E	GPa	210,0	210,0
ECB length	L_{eq}	mm	10,0	10,0
Required moment of inertia – x-x	$I_{x,eq}$	mm^4	1,8896	3,1746
Required moment of inertia – y-y	$I_{y,eq}$	mm^4	317,4603	133,16288
Cross-section type	rectangular			
Cross-section dimension (height)	h_{eq}	mm	1,15	1,56
Cross-section dimension (width)	b_{eq}	mm	14,91	10,09
Behaviour	linear			

* Parameters for shear fasteners (sheet–shear connector) are not used in the building analysis of Chapter 4, as indicated by dash symbols in the respective column; they apply to the unit diaphragm model of Chapter 3 only

As can be seen in the chart, the connection behaviour is close to the design model. Furthermore, it can be noticed that initial flexibility corresponds to the estimated value ("ECCS") as obtained from Davies & Bryan manual [4] or ECCS recommendations [6]. This is proof of the fact that the connector model built for the need of this analysis reflects the design conditions as a compound of theoretical initial shear stiffness with the strength from product documentation.

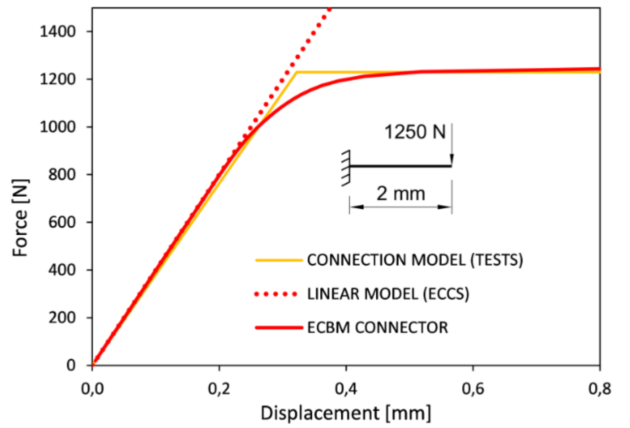

Fig. 7. Comparison of curve generated for the connection modelled using ECBM with the design model obtained from the laboratory experiments [35]

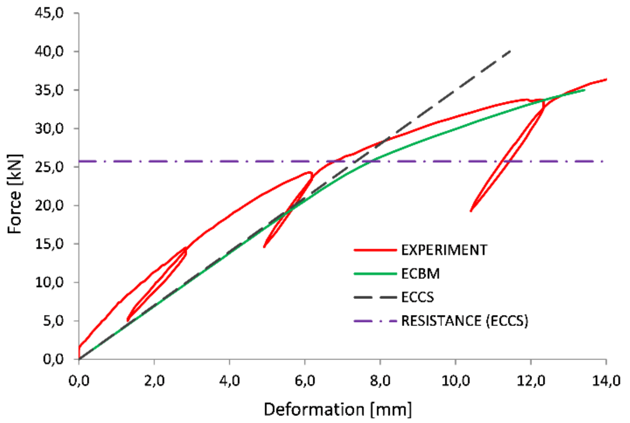

Fig. 8. Results of the unit diaphragm analysis compared to the theoretical calculations using [6] and laboratory outcomes

3.3. The unit diaphragm analysis results

The main outcome from the unit diaphragm analysis is a deflection of the non-fixed rafter under a particular load. Fig. 8 shows the generated curve from the model built using ECBM in comparison to the theoretical calculations [6] and laboratory tests results. The diaphragm behaviour obtained using ECBM works linearly within the first stage (to about 20 kN). This work is different than in the experimental tests, nevertheless it lies on the engineering "safe side" (the theoretical model is less stiff). This difference results from another assumption of the connections stiffness, which has been taken from the ECCS Recommendations, not the experimental samples from the diaphragm (it was not the aim of the experiment). After the linear stage, the deflection rises non-linearly, comparable to the observed behaviour, which is the effect of using non-linear properties of the ECBs. The analysis was carried out until 35 kN was reached. However, the simulation scope was covered from the beginning to about 8 kN above the theoretical resistance, matching the deflection curve from the test. This basic validation demonstrated that ECBM can be used for the prediction of the behaviour of structures considering stressed skin diaphragm action.

4. BUILDING ANALYSIS (COMPLEX VALIDATION)

This analysis is carried out based on past research [20,21,23], rebuilding the numerical model of a steel structure that was analysed in the elastic range only. Then, the results will be compared with those obtained by Gryniewicz, Roberts & Davies [8], where the same building was analysed using advanced models. The structure (15 m span and 20 m length) and the frame loads are shown in Fig. 9.

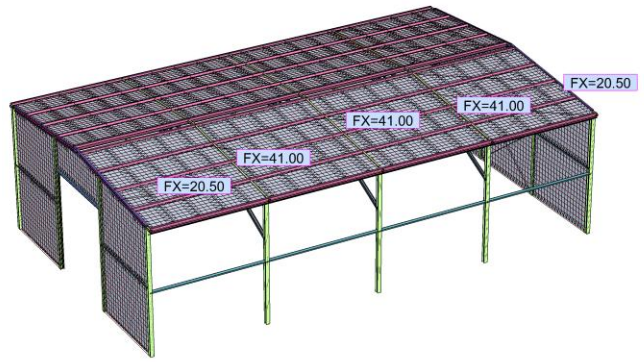

Fig. 9. The frames loads

4.1. Properties of the equivalent cantilever beams (ECBs)

The properties of the connections (**Tab. 1**) are calculated using formulae presented in the second chapter of this paper. Again, unlike the sheet fasteners, the behaviour of the cleats is considered to be linear. Both experiments and advanced numerical analysis results [8] imply that the construction is not influenced by the parasitic diaphragm action (where cleat damage is one of the problems [9]). This conclusion should be verified during the research into the purlin cleat behaviour as well. Additionally, following the referenced analysis assumptions, gables were modelled assuming the estimated flexibility which includes all components, rather than modelling the cladding explicitly.

4.2. Results of the analysis

Results obtained in [8], shown in Fig. 10, are compared to those from the ECBM analysis (dotted curves). Additionally, the case for structure without cladding is added to demonstrate the accuracy and improvement of the proposed approach over conventional methods (neglecting the global stiffening effect of cladding).

The flexibilities of the gables of the covered structure are identical as expected, whereas the behaviour of the internal frames numbered 4 and 2&3 differ. Nevertheless, the values are minor when compared to the scale of structure (Tab. 2). The frame 4 total deformation difference is 1,46 mm. Furthermore, its overall behaviour from the beginning of the simulation deviates. In the model where ECBM has been used, no additional allowances for the flexibility of the bolts in the tension-only roof bracings were applied (in the comparison model they were). These bracings are connected directly to the frame 4. As a result of this simplification (which is common in engineering practice), this frame is stiffer. Additionally, in pair with both approaches modelling differences, it could influence the behaviour of other internal frames. In frames 2&3, the mean difference is 3,18 mm and the parallel deformation of both is

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maintained where the structure geometry is not symmetrical. The yielding history of the seam connections (Fig. 11) also shows a different distribution, however, in the initial stage only. During the later stages, the behaviour tends to be similar to that obtained by Gryniewicz, Roberts & Davies in [8]. Finally, the comparison of the deflected shapes of the roofs, which are similar, is presented in Fig. 12. Considering all the above observations, the outputs from the engineering software using ECBM demonstrate potential and could be useful in practice and research where other tools are not available.

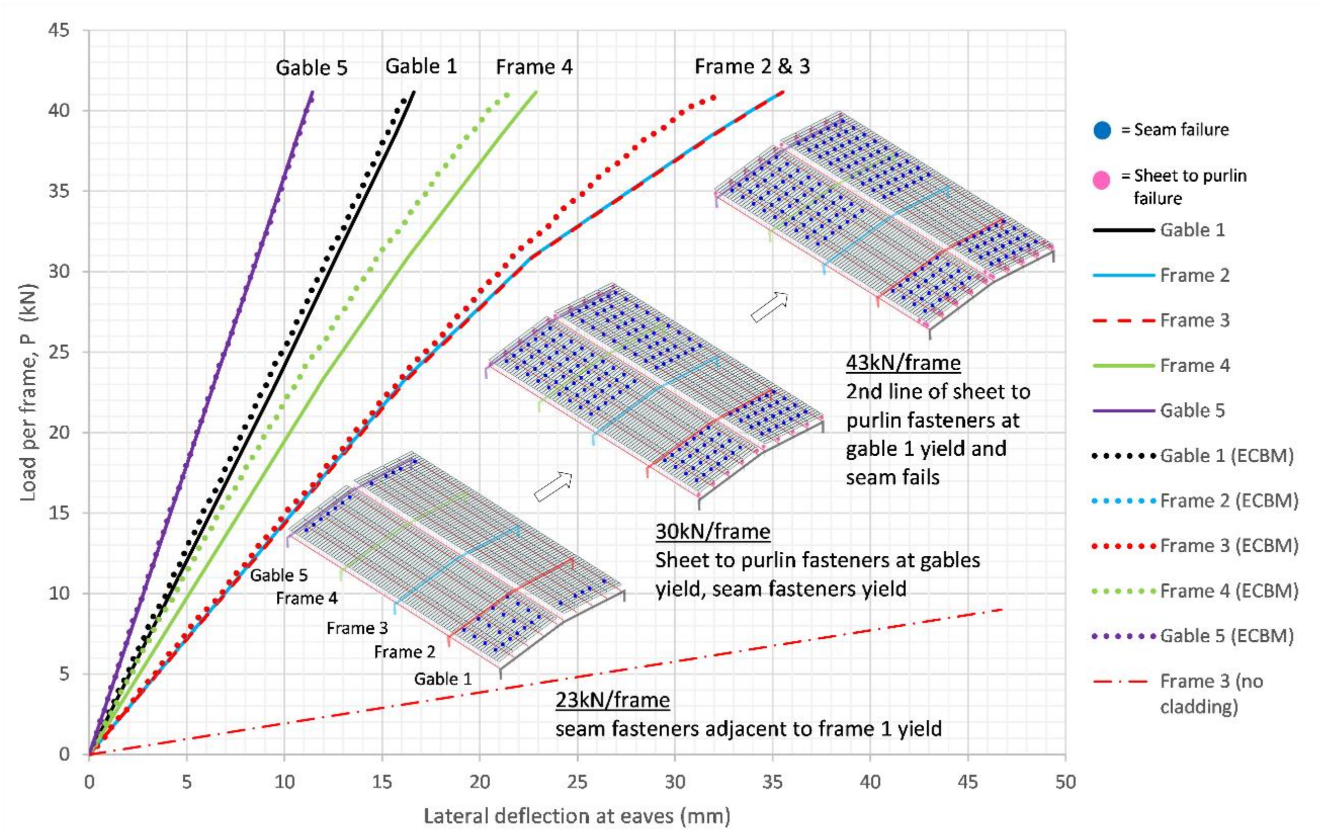
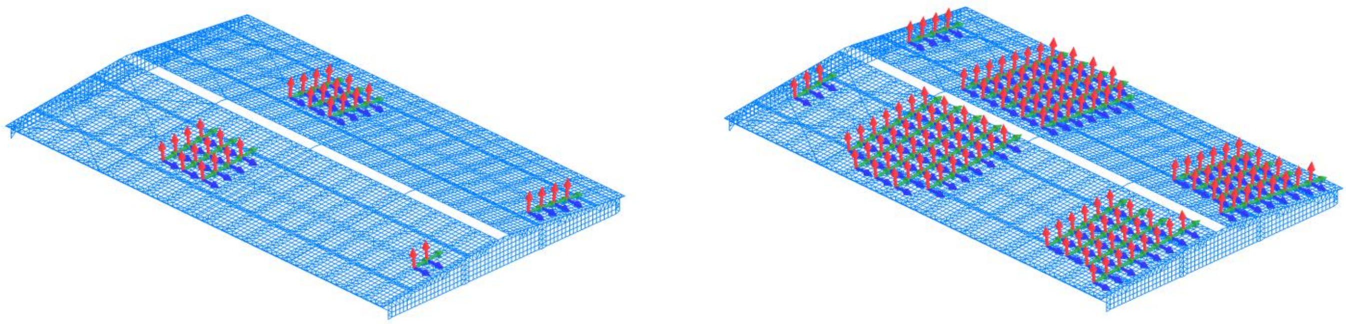


Fig. 10. Nonlinear behaviour of the 3D structure **[8]** obtained using the Roberts & Davies method with added dotted curves resulting from the analysis using ECBM and the case for structure without cladding for comparison purposes

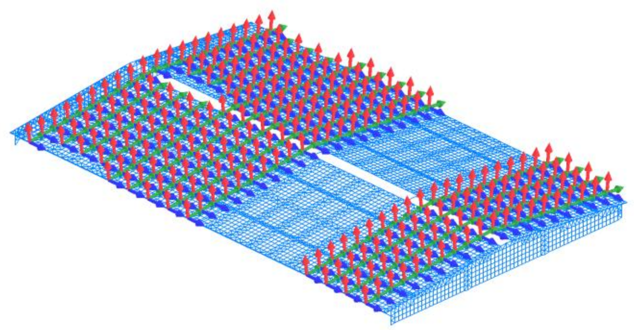
Tab. 2. Comparison of the covered structure frames deformations

Frame		Gable 1	Gable 5	Frame 2	Frame 3	Frame 4
Deformation at 41 kN [mm]	GR&D [8]	16,62	11,44	35,50	35,53	22,87
	ECBM	16,37	11,52	32,33	32,34	21,42
Difference [mm]		-0,25	0,08	-3,17	-3,19	-1,46

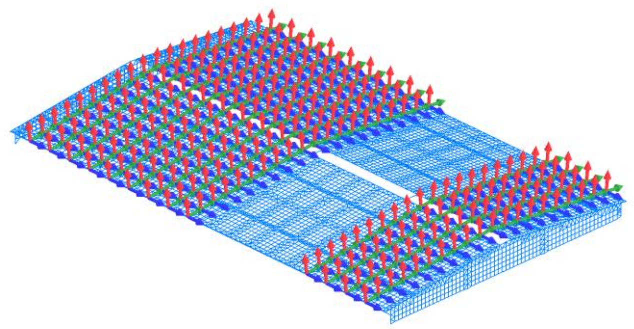


a) 19 kN/frame

b) 23 kN/frame



c) 30 kN/frame



d) 41 kN/frame

Fig. 11. Yielding history of the seam connections

5. ECBM EFFICIENCY AND LIMITATIONS

Experimental tests conducted on a steel building covered with trapezoidal sheeting, followed by simulations using ECBM, demonstrate the significant influence of diaphragm action on the behaviour of the load-bearing structure. As shown in Fig. 10, the horizontal displacement of the building's middle frame is reduced by nearly 90 % — from 25 mm without cladding to 2,7 mm with cladding under a 5 kN load. ECBM effectively simulates these effects, enabling the incorporation of diaphragm stiffness into static models. The key components in the cladding used as a structural member (diaphragm) are connections. A common mistake in complex numerical simulations of buildings is modelling the sheet stiffness solely as a continuous shell, such as over the roof. This approach can lead to an over-constrained model with excessive stiffness, resulting in potentially unreliable outcomes that may compromise structural safety. The use of ECBM eliminates this limitation by explicitly accounting for every joint in the structural system. The challenge may arise during the preprocessing stage, where each connector must be identified and defined. Nevertheless, tools integrated with modern analysis software — such as data generators, scripting modules, or spreadsheet interfaces — eliminate the need for manual geometry modelling. It should also be emphasised that the validation analyses presented in this paper are limited to monotonic static loading; cyclic degradation, hysteresis and fatigue-related damage accumulation are not included in the current formulation.

Earlier concepts of the equivalent cantilever technique [21,23] considered only the elastic behaviour of joints. This simplified approach already yielded promising results. The enhancement introduced by ECBM, through the incorporation of connection non-linearity, has been presented in this paper. It enables diaphragm analysis that accounts for the redistribution of forces through the connections. Furthermore, the method makes it possible to assess the distribution and magnitude of force demand in individual connections and to identify locations where the connection resistance is reached. This is the most important advantage, as in most typical cases, the first damage occurs in the connections.

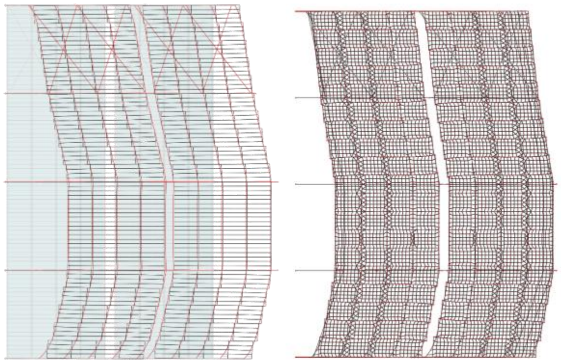


Fig. 12. Comparison of the shapes of the roof deformations

Currently, ECBM focuses primarily on connection modelling, as connections are the key components of structural diaphragms. However, the estimation of the shear strength of trapezoidal sheeting panels – treated as orthotropic plates – still requires further development. The main limitation lies in the behaviour of thin-walled plates in critical areas, such as internal forces concentrations, localized actions, and the edges and corners of the structure. Therefore, special design solutions must be considered to prevent secondary effects, such as instability or cross-section deplanation at supports. This aspect is already addressed in practice literature [4,6], partly in response to the limitations of analytical methods used to estimate diaphragm performance (e.g. edge stiffening members, connectors spacing limits or ensuring sufficient vertical load capacity prior to horizontal loading). These recommendations must be taken into account when implementing ECBM in structural design.

Ensuring sufficient vertical load capacity prior to horizontal actions results from the assumption that strength analysis for vertical and horizontal loads can be treated separately. This assumption originates from Eurocode [7], which states: "all sheeting that also forms part of a stressed-skin diaphragm should first be designed for its primary purpose in bending". At this stage, structural design using the ECBM must adhere to this rule, as the effects of combined loading (vertical and horizontal) were not considered. The ECBM has been developed in accordance with current practices established in the literature and codes of practice to ensure direct compliance [29]. This is the reason why orthotropic plane-stress elements (which do not include out-of-plane work) were used for the simulations. Nevertheless, the ECBM is not limited to a plane-stress type of connected continuum. It is possible to use different types of surface models under various loading conditions. However, such configurations must be verified separately as a new adoption of the method.

In addition to addressing the modelling details and their inherent limitations, differences between ECBM and more advanced approach, particularly the Roberts & Davies method [8], were identified. While some discrepancies may arise from the modelling of the bearing structural components (frames), the most significant factor appears to be the ECBM use of equivalent material properties coupled with the selected cross-sections. This feature can have an influence on the modelling of purlin-rafter connections (cleats, which have different stiffnesses and behaviour in various directions). Recent research [12], [23] proved that for modern, especially large

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warehouses, this issue cannot be ignored. The present implementation of the ECBM does not attempt to reproduce the full nonlinear behaviour of purlin-cleat connections. In the validation cases analysed in this paper, these connections were represented by linear stiffness values, consistently with the available classical assumptions and with the reference analyses used for comparison. This simplification was considered acceptable because the response of the investigated structures was not governed by purlin-cleat nonlinearity, sign-dependent stiffness or cleat failure. Nevertheless, recent research indicates that the behaviour of modern cleat configurations may be direction-dependent, configuration-dependent and sensitive to cyclic loading. For this reason, the present nonlinear extension should be understood as being limited to sheet-to-purlin and sheet-to-sheet fasteners, while the development of experimentally or numerically validated representations of purlin-cleat nonlinearity, directional asymmetry and fatigue-related effects remains an important subject for future work.

Considering above findings, a contemporary approach involves the validation process proposed by Roberts & Davies [36], shown in Fig. 13, which offers the a framework to follow to produce validated models. The test set-up shown in Fig. 14 offers a potential basis for laboratory tests when the cleat behaviour is of significance due to the inclusion of the realistic interface between the primary structure (rafters) and cladding system. Thus, these findings directly inform the ongoing refinement of the ECBM methodology.

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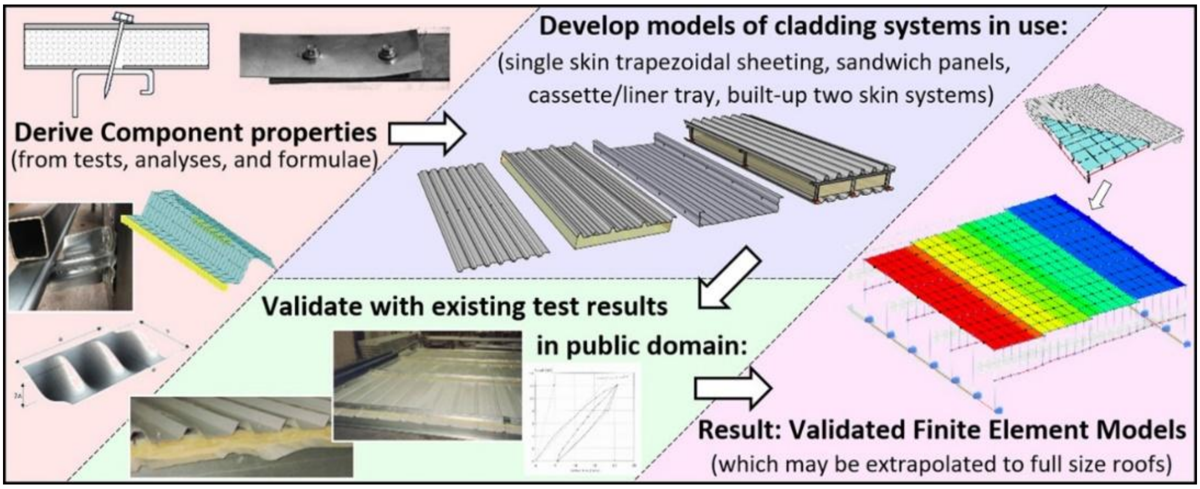

Fig. 13. Graphical abstract demonstrating validation process [36]

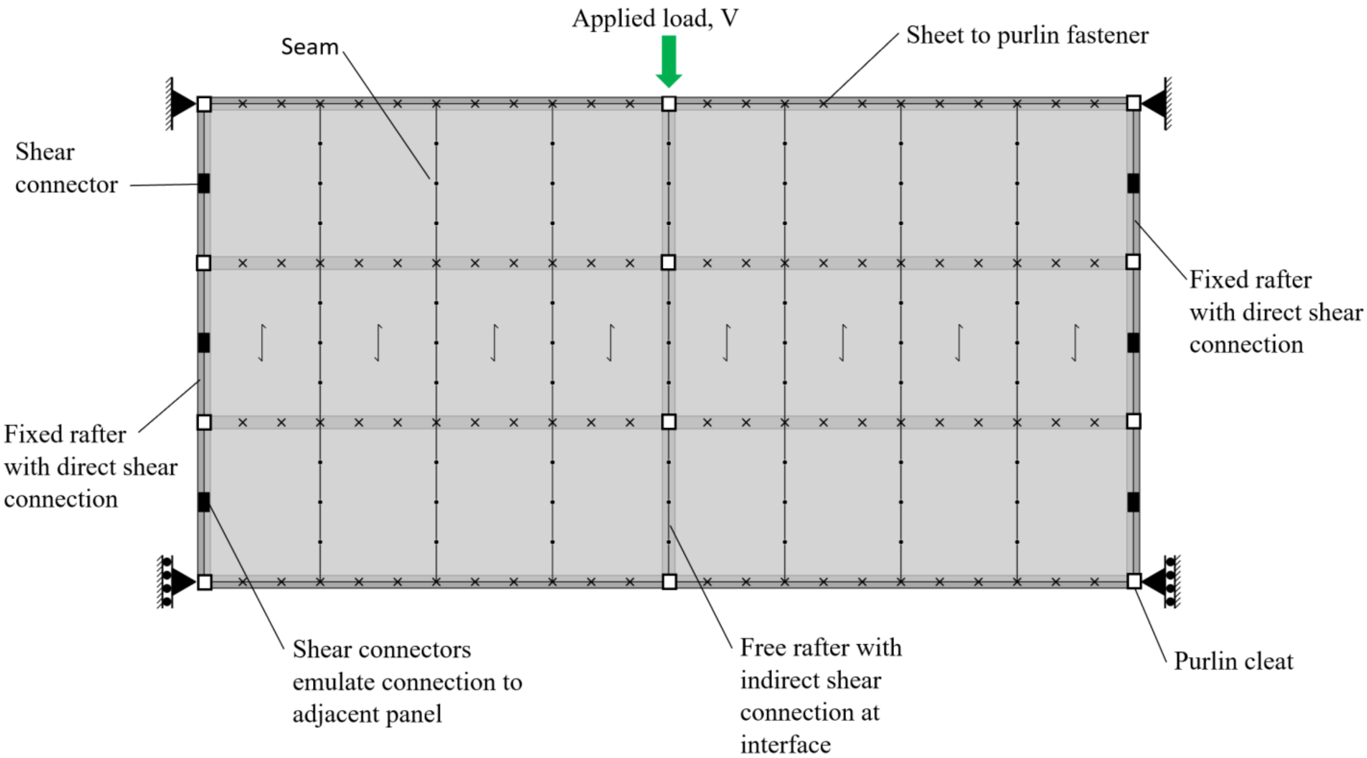

Fig. 14. Test set up with a realistic "interface" by Roberts & Davies [36]

6. CONCLUSIONS

The simplified methods of buildings analysis, including stressed skin diaphragm action, developed in the past [21,23] have been revisited. A novel improvement of the available methods, termed the "Equivalent Cantilever Beam Method" (ECBM) is proposed. The method has been validated by means of a hierarchical methodology. Derivation of the individual connector properties were checked with a single ECB behaviour. Next, a model of a complete shear cantilever panel was validated using experimental results. Finally, outputs from advanced analysis of the building were comparable with the advanced numerical method by Roberts & Davies, developed and validated recently in [8].

Experimental tests and numerical simulations confirm that diaphragm action has a substantial impact on the global behaviour of steel structures, significantly reducing horizontal displacements and improving overall stiffness. The Equivalent Cantilever Beam Method (ECBM) effectively incorporates these effects into static models by explicitly considering joint behaviour and connection non-linearity, enabling more accurate force redistribution analysis. Despite these advantages, limitations remain, particularly in modelling trapezoidal sheeting as orthotropic plates and assuming linear properties for cleat connections. Further development is required to address these aspects and ensure reliable modelling of complex interactions between cladding systems and primary structural components.

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