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Advanced design and optimization of steel portal frames

Petr Hradil, Matti Mielonen and Ludovic Fülöp

Summary. This study brings new insights into the advantages of using more sophisticated design methods for steel portal frames (e.g. geometrically and materially nonlinear analysis with imperfections or GMNIA, and the general method introduced in Eurocode 3), compared to the commonly used member checks with interaction formulae. The differences between the design alternatives are discussed, focusing on assessing lateral-torsional stability, and the potential benefits of using advanced shell model instead of widely used beam elements. In addition to the advanced design methods, the topic of shape optimization of frames was explored using real-coded genetic algorithm (GA).

The developed optimisation tools highlight the possibility of using GA in everyday design in the future. The results of the study clearly point to the advantages of using advanced modelling, e.g. GMNIA, instead of the classical member checks. While both methods are accepted by the current steel design code EN 1993, using GMNIA can result in important savings, because it eliminates some of the conservativeness brought in by the unavoidable simplifications of the other methods. The experience shows that using complex 3D models is possible with current computational capabilities.

Key words: portal frames, genetic algorithm, general method, lateral-torsional stability

Introduction

Portal frames for industrial buildings have been extensively studied because of their widespread use. The improvement of the design methods for portal frames is one of the recurring topics in the field of steel structures. Due to the large number of similar framed structures, the desire to "automate" the design and manufacturing process was popular from the very early stage [2]. As Dowling et al. [2] noted, there are two design tendencies when trying to achieve more economical solutions: (a) to use compact hot-rolled sections and exploit the advantages of plastic design and (b) to use slender built-up sections with the most advantageous distribution of the material but keep the design in the elastic range. The second option usually leads to slender structures, and therefore stability becomes the main concern of the designer.

In case of side rails and purlins, the elastic design with very slender, Class 4 cross-sections (i.e. cold-formed profiles) tends to be more economical solution compared to the plastically designed hot-rolled continuous purlins. The better distribution of material in cold-formed profiles is clearly offsetting the higher costs of fabrication. Moreover, modern coating technologies also allow the use of material thicknesses in the range of millimetres and below, without fears of corrosion.

Dowling et al. [2] predicted that "an equivalent development towards slender construction in main frame design will lead to similar economies to that achieved with secondary elements". This conclusion has been supported by the experience in the US [3], Canada and some European countries. However, the situation in Europe is not as clear as in the US where the slender tapered solution almost completely replaced the hot-rolled frames due to the mechanised fabrication of tapered elements. Several European design documents [4], [5], [6], [7] including EN 1993 [1] are more focused on plastic design of frames. In fact, both solutions coexist on the market in many countries.

The economical efficiency of the two solutions depends on several related factors, such as fabrication cost, transport cost, labour cost. Dowling et al. [2] reported weight saving in range of 30% in favour of slender tapered frames. However, it was debated what this means in cost saving [8]. Other aspects of using the welded tapered solutions were also discussed, with important focus on the question of lateral stability of beams and columns [8].

The present study proposes to update the discussion on optimal frame design: (1) by using design methods for stability from the EN 1993 [1], (2) by implementing advanced analysis methods, which are not currently employed by design offices but have potential for replacing current tools, and (3) by introducing optimization tools based on Genetic Algorithms (GA's) to find optimised geometries.

Design methods

The study includes several methods for portal frames design to resist loads in fundamental and seismic situations, using the limit states conditions from the EN 1993 [1] and EN 1998 [9]. To implement those design methods in optimization, Abaqus and Excel based plug-ins were developed. Both plug-ins allow automatic generation of computational models and automatic result evaluation. The results of the design checks are expressed as a minimum load amplifier of the design loads to reach the ultimate (ULS) or serviceability (SLS) limit state criteria. The vertical serviceability limit corresponds to an apex deflection of span/200, whereas the horizontal serviceability limit is height/100; derived in a simplified way from EN 1998 [9].

Method 1: Geometrically and materially nonlinear analysis on imperfect structure (GMNIA)

The most computationally expensive method is the straightforward numerical calculation of 3D shell model of the frame (Figure 1). No special checks for out-of-plane stability are needed in such method, because the calculation is materially and geometrically nonlinear and is taking into account appropriate initial imperfections. Bow imperfections are inserted from a preliminary eigenvalue analysis, using the first positive buckling shape of the model, scaled to the EN 1993 recommended amplitude (i.e. using Table 5.1 from EN 1993). Sway imperfections are also applied to the model according to EN 1993 rules. The material was modelled using a bilinear stress-strain relationship, including a moderate strain hardening calculated for each used steel grade.

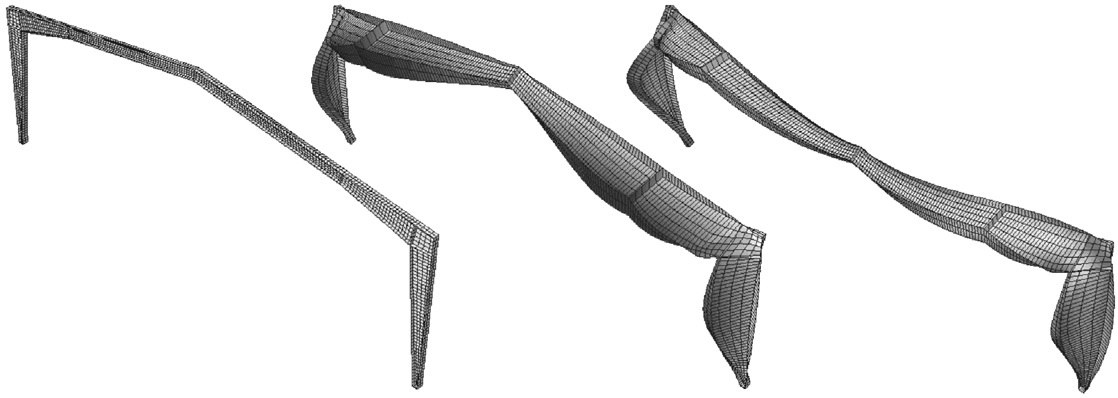

Figure 1 Global nonlinear analysis calculation steps: Undeformed model (left), First buckling shape (middle), Deformation during nonlinear analysis with increasing vertical loads (right - scale: 20)

Method 2: General method

This method takes into account out-of-plane stability with a global reduction factor. According to §6.3.4 of EN 1993 [1], the resistance of the frame is checked using the following condition:

$$\frac{\chi_{op} \cdot \alpha_{ult,k}}{\gamma_{M1}} \geq 1.0\,, \tag{1}$$

where $\alpha_{ult,k}$ is the minimum load amplifier of the design loads to reach the characteristic resistance of the most critical cross-section without out-of-plane buckling effect. This value is obtained from the nonlinear analysis with gradually increasing loads of a simple 2D beam model of the frame. However, even the 2D model is materially and geometrically non-linear and it includes in-plane bow and sway imperfections (but not out-of-plane sway imperfections).

The out-of-plane buckling reduction factor χ_{op} originates from the critical amplifier $\alpha_{cr,op}$ of the design loads to reach the elastic critical resistance with regards to lateral or lateral torsional buckling. Because of the particularly difficult analytical expression of critical load of frames which are made from eccentrically supported members with variable cross-sections, the method is using the 3D shell model of the frame for the evaluation of the critical amplifier.

Method 2 has the advantage of faster numerical calculation compared to Method 1. The computational advantage results from the use of the simplified 2D beam model for the nonlinear part of the analysis, and the use of the 3D shell model only for the evaluation of the critical loads for out of plane buckling.

Method 3: EC3 interaction formulae

Linear structural analysis together with design using cross-sectional checks to express limit states conditions are the commonly used methods well described in EN 1993 [1]. However, not all of the EN 1993 rules are applicable to elements with variable cross-section and eccentric lateral restraints. Therefore, it was necessary to implement other theories in the calculation. Even though the method is the most computationally effective, it is more conservative, especially in its out-of-plane stability approach where no lateral supports of compressed flanges are used by default.

The Method 3 uses the following modifications of the calculation described in EN 1993 [1]:

In-plane critical amplifier

While the critical load was a result of linear eigenvalue analysis in Method 2, the analytical approach is adapted in this case. The calculations take into account the presence of the axial force in the rafters and utilize the formulas proposed by Davies [10].

Lateral torsional buckling of eccentrically restrained tapered member

The critical moment in our calculation is the smaller value of critical moment of restrained member between fork supports according to the Appendix F of the SCI Technical Report [7], and elastic critical moment of unrestrained part of member between purlins or side rails.

$$M_{cr} = \min\left[\left(\frac{1}{m_t c^2}\right) M_{cr0}, \quad C_1 \frac{\pi^2 E \cdot I_{z,\min}}{a^2} \sqrt{\frac{I_{w,\min}}{I_{z,\min}} + \frac{a^2 G \cdot I_{t,\min}}{\pi^2 E \cdot I_{z,\min}}}\right], \tag{2}$$

$$M_{cr0} = \left(\frac{i_0^2}{2e_z}\right) N_{cr,TFB}. \tag{3}$$

Here m_t is equivalent uniform moment factor adapted from [7], $I_{y,min}$, $I_{z,min}$, and $I_{w,min}$ are sectional properties of the shallow end, a is the distance between lateral supports (purlins, side rails), $N_{cr,TFB}$ stands for critical axial load of torsional flexural buckling of eccentrically restrained member, C_1 is the moment gradient factor, e_z is the distance from shear centre to lateral support either at shallow end or at deep end of tapered beam.

When the relative slenderness is higher than 1, equivalent section factor c is also 1 and the minimum $M_{cr,0}$ was used, which comes from the deepest end. In all other cases shallow end properties and following expression for equivalent section factor are used:

$$c = 1 + (c_0 - 1)\sqrt{L_h / L}, \tag{4}$$

where L_h and L are the lengths of haunch and the whole member respectively and the basic equivalent section factor c_0 was adapted from [7].

Major axis flexural buckling of tapered member

It is very conservative to apply the standard formula for calculation of the critical load of the tapered member using the shallow end's sectional properties. A more accurate option is the approach proposed by Šapalas [11] used in Method 3, where critical loads are based on the deep end cross-sectional properties and reduced by α_n factor.

Analytical results

To demonstrate the differences between the three design methods, the results of calculation of one particular frame are presented in this chapter. A welded-tapered frame with pinned supports was calculated with the parameters presented in Figure 2 and Table 1. The loads from the most critical fundamental combination of dead weight and snow was redistributed into a set of concentrated forces at the purlin/rail connecting points. The factored value of distributed load used for the ULS and SLS calculations checks was 1638 N/m^2 and 1130 N/m^2, respectively.

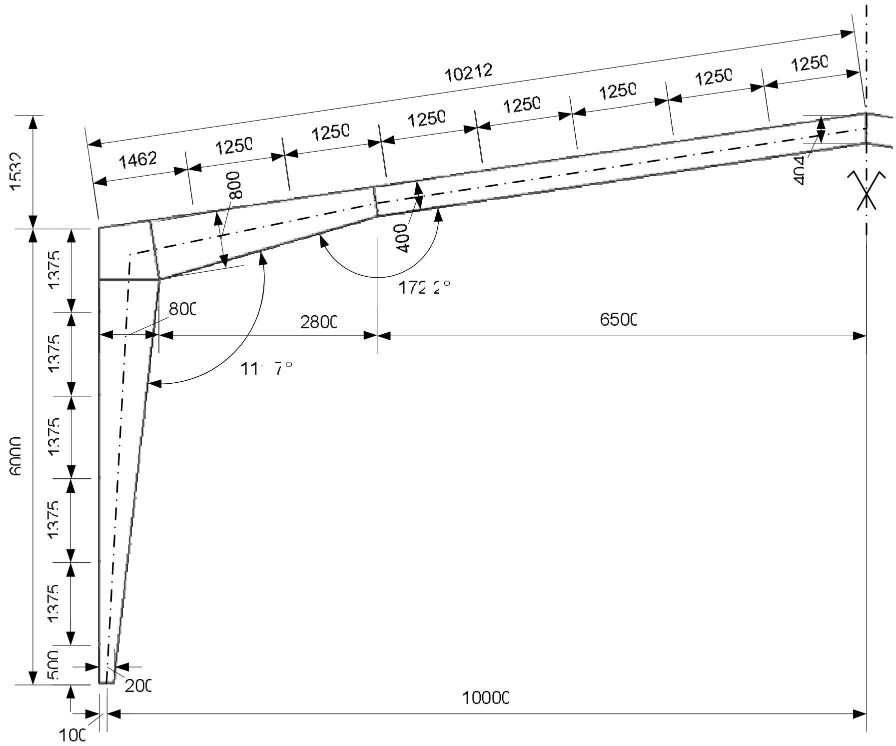

Figure 2. Geometry of the selected welded-tapered frame symmetrical part

Table 1. Cross-sections, material and loading of the selected frame

Flange thickness	10 mm	Web thickness	8 mm
Flange width	260 mm	Haunch length	3.6 m
Avg. purlin spacing	1.25 m	Avg. side rail spacing	1.38 m
Purlins/rails eccentricity	120 mm	Steel quality	S275
Char. dead load	380 N/m^2	Char. snow load	750 N/m^2
Dead load safety factor	1.35	Snow load safety factor	1.5

The frame satisfies the ultimate and serviceability limit states conditions with all three design methods, but with different levels of conservativeness. (Table 2). The required computing times for carrying out the check, are also given in Table 2.

Table 2 Calculated load-bearing capacity of the frame at ultimate (ULS) and serviceability (SLS) limit states and the calculation speed (tested on Pentium 4, 3 GHz with 1 GB RAM)

	Method 1	Method 2	Method 3
ULS Capacity	2800 N/m^2	2272 N/m^2	1761 N/m^2
SLS Capacity	3088 N/m^2	3359 N/m^2	3410 N/m^2
Calculation speed	284 sec.	125 sec.	2 sec.

Figure 3 represents load-displacement curves produced by different methods. In case of Method 1, the resistance of the frame can be estimated directly from the curve. Because the model includes both in-plane and out-of-plane imperfections, and the analysis method is materially and geometrically nonlinear, the effect of buckling is implicitly taken into account. Therefore there is no need to affect the resistance by buckling factors, while safety factors for material and loads are included in the model. One question, when interpreting the curve, is what to consider on the curve as resistance limit. In our case, the resistance limit is considered the reaching of the first yield, because the frame is slender. However, this might supply conservative results if the frame is made of compact members or if a stress concentration causes local yielding. In these cases the resistance can be interpreted according to methods recommended by ECCS [12].

In case of Method 2, the curve includes only in-plane imperfections, both sway and bow, and the analysis is materially and geometrically nonlinear. The value of $\alpha_{ult,k}$ to be used in equation 1, is estimated from this curve as corresponding to the point where plastic strain reaches five times the yield strain in any cross-section. However, $\alpha_{ult,k}$ is further reduced due to out of plane buckling with an extra reduction factor (χ_{op} in equation 1) calculated from the overall slenderness. The straight line corresponding to Method 3 is the result of a linear analysis which includes sway imperfections.

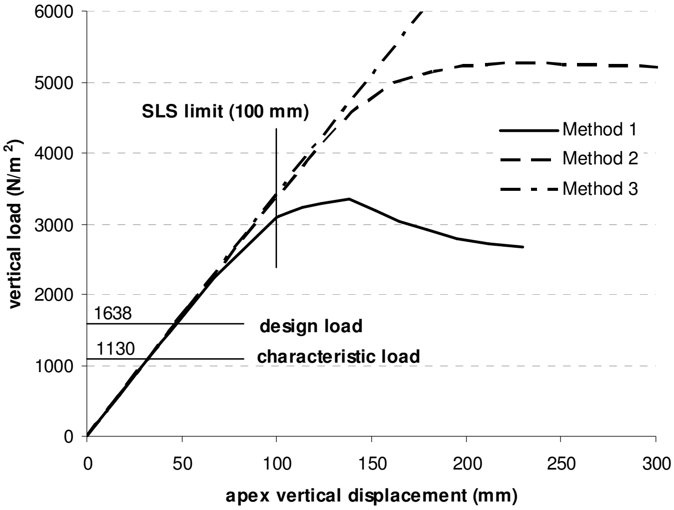

Figure 3. Load-displacement curves; comparison of different methods: Method 1 (solid line) including all buckling and imperfections, Method 2 (dashed) including only in-plane buckling and imperfections, Method 3 (dash-dot) including only initial sway imperfections

Optimization method

The steel portal frame optimization problem is formulated below:

$$\begin{aligned} &\text{Minimise} && f_w(\bar{x}) \\ &\text{subject to} && g_{ULS}(\bar{x}) \geq 0, \\ & && g_{SLS}(\bar{x}) \geq 0, \\ & && \bar{x} \in \{x_b, x_c, x_h, x_b\} \text{or} \\ & && \bar{x} \in \{x_h, x_{tw}, x_{tf}, x_B, x_H, x_{Hh}\} \end{aligned} \tag{5}$$

Here, $f_w(\bar{x})$ refers to optimization searching for a minimum weight. The possible solutions are checked against two main constraints: g_{ULS}, where ULS refers to ultimate limit state check and g_{SLS}, where SLS refers to serviceability limit state check. The variable vector $\bar{x}$ comprises beam profile x_b, column profile x_c and haunch ratio x_h variables, if the frame is assembled of hot-rolled sections. When welded-tapered sections are optimized, number of possible variables increases. The variables available for optimization are profile flange thickness x_{tf}, profile web thickness x_{tw}, profile width x_B, profile height x_H and height of the profiles at the haunch x_{Hh}. Theoretically, these variables could be selected separately for the beam and column, however, it is reasonable to limit the number of different plate sizes. Therefore, only height of the lower end of profiles can be different in the beam and column, which leaves six variables for optimization ($x_{tf}, x_{tw}, x_B, x_{Hc}, x_{Hb}, x_{Hh}$). Each variable has boundaries coming from physical restrictions (e.g. haunch ratio 4-11) and readily available parts (e.g. common plate sizes 4, 5, 6, 8, 10, 12, …).

The problem is both nonlinear and discrete, which causes difficulties for classical direct and gradient-based optimization methods. On the other hand, genetic algorithms (GAs) are successfully applied in the field of structural optimization [13], [14], [15] and [16]. GAs have several advantages, such as possibility of parallel computing, easy handling of multiple variables, and straightforward coding practice. Genetic algorithm is a procedure which tries to mimic the natural evolution process. After an initial population is created and analysed, the fitness of each individual is evaluated. Then a new population is created by favouring the fittest individuals and by combining the properties of the population members using genetic operators, such as crossover and mutation. GA proceeds iteratively towards the optimal solution by creating a new population using the properties of the previous one. Elitism ensures that the best solution is kept during all of the genetic operations, and while no proof of convergence in finite time exists, good results are found in a reasonable time.

A real-coded genetic algorithm (RCGA) can handle discrete and real variable types easily and has been chosen for this problem. With RCGA, the coding-decoding characterizing binary-coded GAs is also avoided.

Optimization literature provides a large catalogue of different selection, crossover and mutation operators that can be combined to create a GA suited for the problem at hand. In this paper, the well known simulated binary crossover (SBX) [17] and parameter based polynomial mutation operator [13] are utilized. The crossover operator has a self-adapting behaviour, which favours creating children near to parents, when the parents are near to each other in the variable space. The basic behaviour of the genetic algorithm is enhanced by two methods developed to improve the steel portal frame optimization:

- a local search method is creating individuals very similar to the current elite individual found, which ensures that the local optimum is found with very high probability;
- the diversity of the population is maintained by using so called diversifying operator, which introduces new genetic material to the population preventing premature convergence to the local optimum [16].

In terms of optimization objective function, it has been opted to concentrate on easily measurable performance parameters, such as weight, with the flexibility to expand results to more financially oriented targets (e.g. price).

The algorithm can be used with all of the design methods and the flowchart of the optimization combined with design methods is presented in Figure 4.

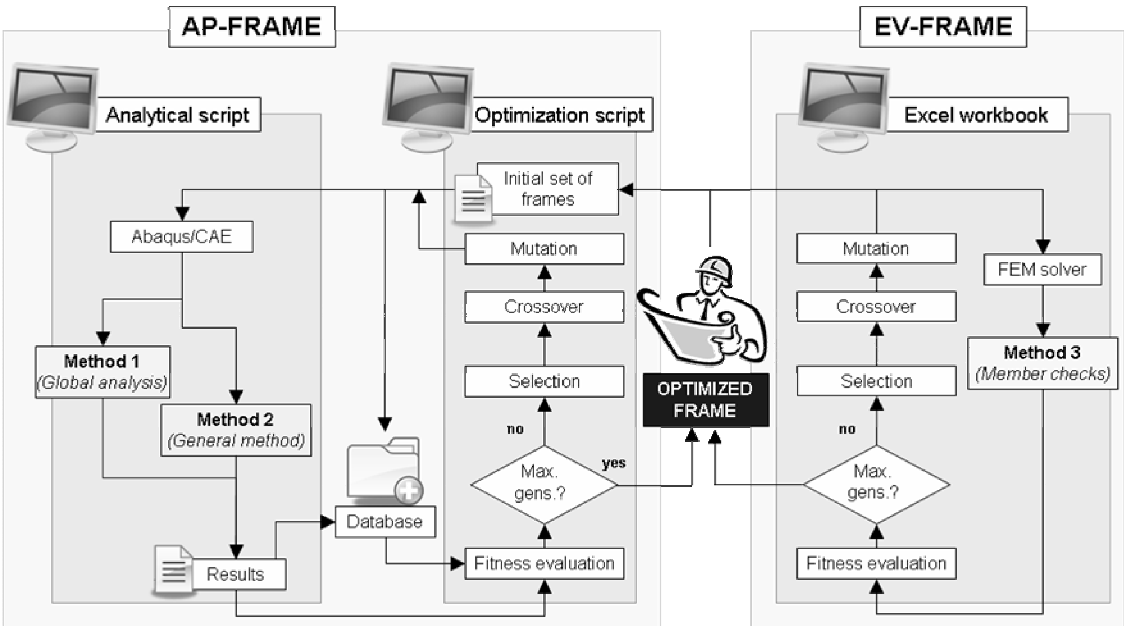

Figure 4 The basic diagram of two optimization tools developed at VTT.

AP-Frame contains the Abaqus based design methods and optimization, whereas EV-Frame refers to the design and optimization developed in a Microsoft Excel workbook. In both cases, the GA runs for predetermined number of generations, and the best configuration found is given as an output. Figure 5 presents five example optimisation runs of the same frame configuration, which shows how the weight of the elite individual typically decreases during the genetic algorithm optimisation. In this example, eight variables were optimized in a welded-tapered frame with population size 20 and maximum 50 generations.

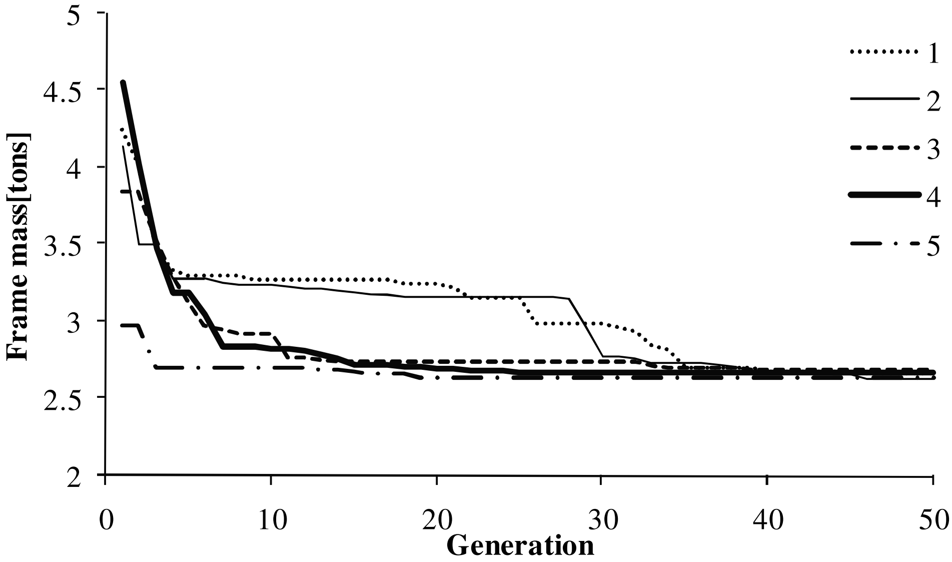

Figure 5. Elite value fitness development in five optimisations of the same frame configuration with the genetic algorithm.

Typically, the weight of the elite frame in the population decreases sharply during the first generations, and then the search focuses around the elite individual refining the solution. Sudden drop in fitness occurs usually when the algorithm finds a better alternative shape or size of the frame elements, meaning that many variables can change drastically.

Benchmarking the design and optimisation methods

In order to calibrate the tools, the results of the optimization study by Horridge et al. [18] have been replicated. A couple of important points in this study must be taken into consideration in order to understand the extent of calibration presented here:

- The configurations considered were based on the Universal Beam (UB) range of class 1 profiles allowing for plastic analysis. In line with British practice, first order plastic analysis was used for the design (i.e. no imperfections, no second order effects and no buckling were considered).
- The frames were considered pinned. Therefore the structures cannot develop a plastic mechanism with several plastic hinges.
- The eaves height was increasing together with the span and it should be noted that the steel consumption (Figure 8) was considerably affected by this fact.
- No serviceability checks were reported in the original study [18].
- The results are not based on any formal optimisation method, rather it was attempted to find optimal configurations based on engineering experience. The roof angle and the haunch length were fixed in advance.

The cases reported by Horridge et al. were reproduced using the EV-frame and AP-Frame tools. It was impossible to fully recover all original design assumptions because some incompatibilities exist between the old British codes and the Eurocode methods. Therefore, a few equivalent assumptions had to be used. Namely:

- The AP-Frame and EV-Frame tools were forced to neglect lateral torsional buckling. This was achieved by setting buckling reduction factors to 1, and all out-of-plane imperfections to 0.
- The safety factors of loading were modified to match those used by Horridge et al. [18], i.e. $\gamma = 1.7$ for all loads.
- The basic wind speed of 46 m/s (3 seconds average) reported to be used with the British code, was replaced with 15.5 m/s (10 minutes average) wind speed compatible with EN 1991 and producing the same total horizontal load. It should be noted that the factors controlling the wind pressure on the roof are also different in the two codes, but this difference has not been eliminated.
- The steel grade 43 reported by Horridge et al. [18] has been replaced by S275, with the same yield stress.
- Instead of the classic plastic method, allowing of the successive forming of plastic hinges, the calculation uses linear and nonlinear elastic analysis with plastic sectional properties, as described by EN 1993 [1]. This method could be adopted because frames are pinned and they cannot create successive plastic hinges.

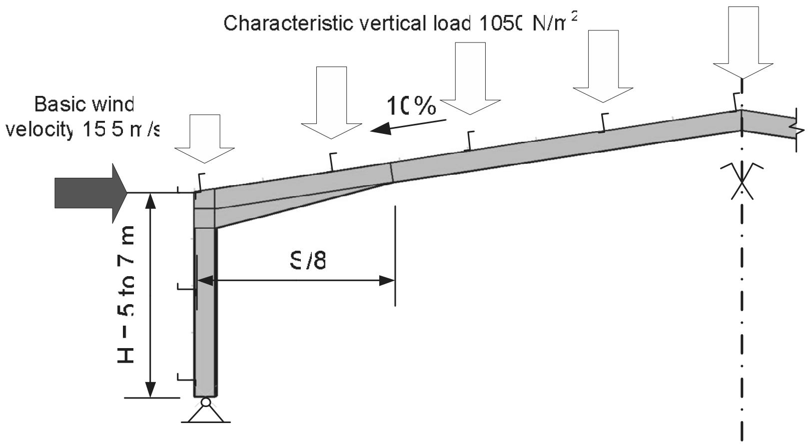

Figure 6. Geometry and loading of hot-rolled frames

The optimisation used by Horridge et al. [18] was emulated with the above assumptions, and the original configurations were re-calculated using EV-Frame and AP-Frame tools. These configurations had a ULS utilization factor in the range of 0.93-1.22 with EV-Frame and 0.88-1.13 with AP-Frame (Figure 7). It can be concluded that both software tools were able to recreate the initial set with good accuracy. EV-Frame is slightly more conservative, predicting an average utilization factor of 1.06, suggesting that the frame proposed by Horridge et al. [18] would fail using linear elastic analysis with plastic sectional properties. AP-Frame is less conservative with nonlinear elastic analysis based on equivalent plastic strain check to identify the plastic hinge. The average utilization factor was 0.98 and the method predicted problems with only few of original configurations.

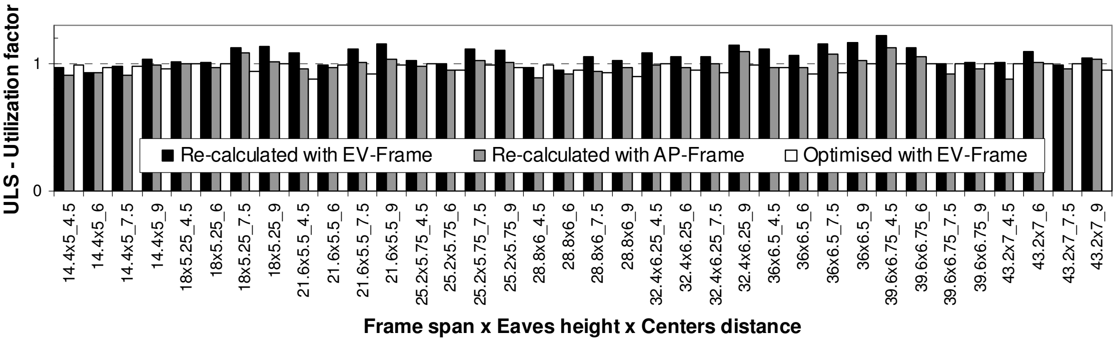

Figure 7. Recalculated and optimized cases utilization of ULS checks

The set of frame configurations used by Horridge et al. [18] have been optimized using the EV-Frame, with exactly the same constraints (Figure 7) using beam and column profiles as variables. While the original optimisation proved to be very good, we found that marginal improvements are possible even in this very rigorously studied set of frames.

For instance, the EV-Frame optimized frames had a slightly different balance between beam and column compared to the original frames. On average, plastic modulus of beams increased by 10% while plastic modulus of columns decreased by 7%, suggesting that weaker beams and stronger columns, compared to the usage of Horrige et al. [18], are the optimal configuration.

In Figure 8, the steel consumptions are given for the frame distance 6 m, also considered in the original study. These consumptions were calculated using a single frame mass without any additional steel elements e.g. bracings, purlins, longitudinal beams etc. It can be observed that the original configurations by Horridge et al. [18] and the optimized ones by EV-Frame and AP-Frame have very similar weight.

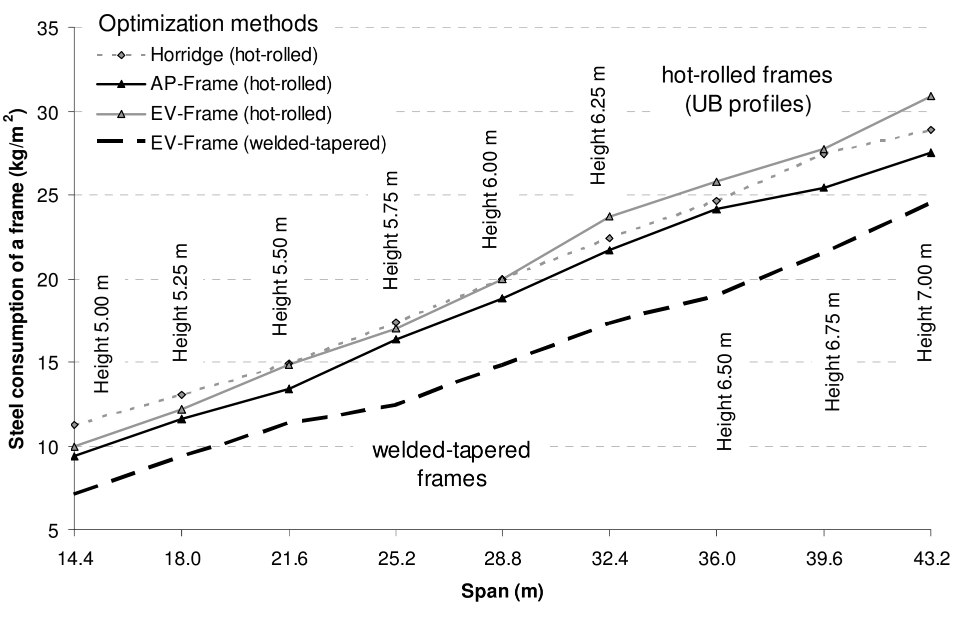

Figure 8. Comparison of different design and optimisation methods

Optimization of welded-tapered frames

The optimization of hot-rolled frames, mentioned in previous chapter, was based on two variables with limited amount of possible combinations. The result of this simple task, where haunch ratio and roof pitch are fixed, could be found easily using other methods, including calculation of all 5184 possibilities. The most important feature of our optimization tools is the ability to find the lowest mass of welded-tapered frames where millions of combinations are possible.

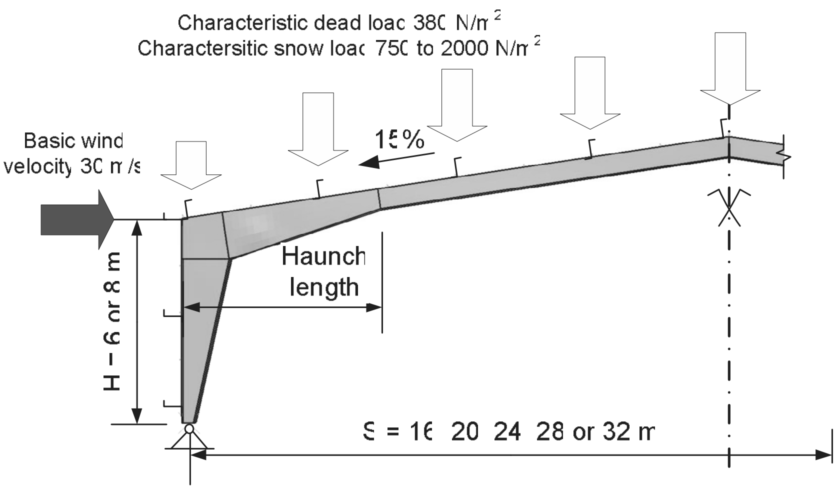

Figure 9. The geometry and loading of the studied welded-tapered frames

The first study based on our optimization tools aims to demonstrate the relation between frame mass (or steel consumption per m^2) and frame span (Figure 10) with different snow load and eaves height. The variables optimized are haunch ratio, flange thickness, web thickness, height of the profiles at the eaves, width of the profiles, height of the beam at the top and height of the column at the base. Assuming that there is no effective lateral support of compressed flanges, boundary conditions exists only at the external flanges in 3D shell model. Also the wind load is considered as a secondary variable action creating pressure on upwind side and uplift on downwind side according to EN 1991[19] (Figure 9).

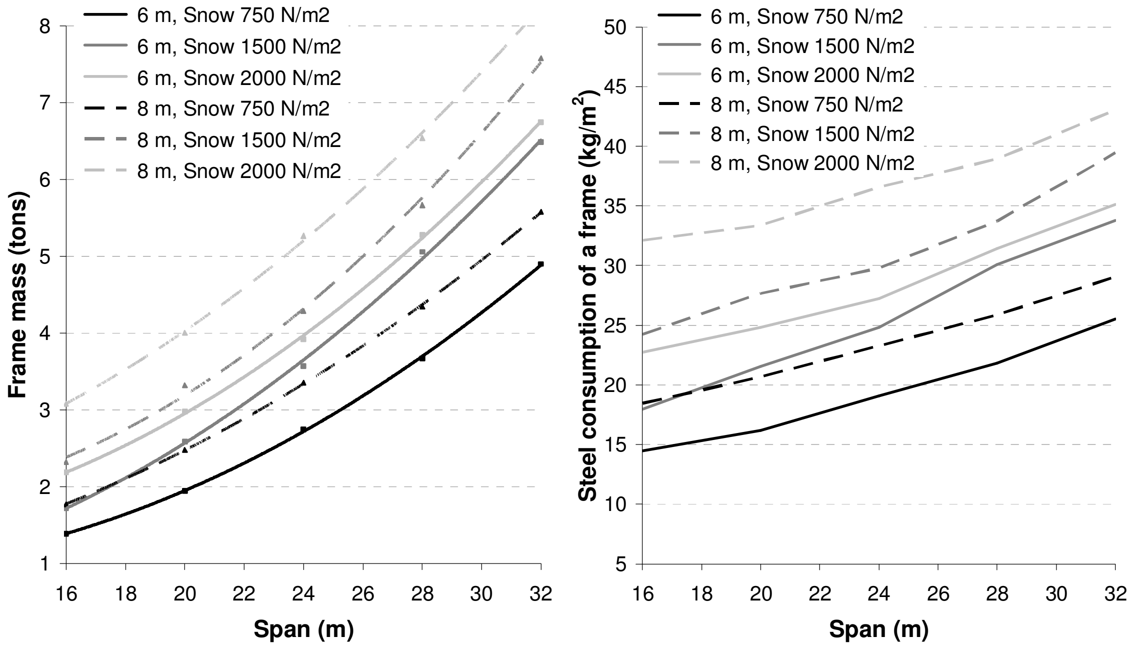

Figure 10. The effect of different snow loads on welded-tapered frames 6 m high (solid line) and 8 m high (dashed). The load is represented as a characteristic value on flat roof (e.g. 2000 N/m^2 corresponds to basic snow load 2500 N/m^2 on the ground)

The lateral buckling of slender cross-sections is significantly reducing the load-carrying capacity of the frame, and it is common practice to use lateral restraints connected to the inner compressed flanges to suppress this negative effect. The lateral support is usually considered to be strong enough to completely suppress out-of-plane buckling.

However, the most frequently used type of lateral restraint, the diagonal stay, transfers lateral loads by bending of purlins, and tends to be less effective when cold-formed purlins are used. In order to evaluate the behaviour of such frames, the simple 2D beam model is not sufficient. In the following study, 3D model in AP-Frame optimization tool expands by added purlins and lateral restraints (Figure 11). The purlins are modelled as beam elements with 2 mm thick Z150 cross-section, and lateral stays are simplified as rigid truss elements. The diagonal stays exist only in the corner region of the frame supporting the whole length of the haunch and one third of the column. It has been tested that adding more supports has no further effect on lateral stability of the frame.

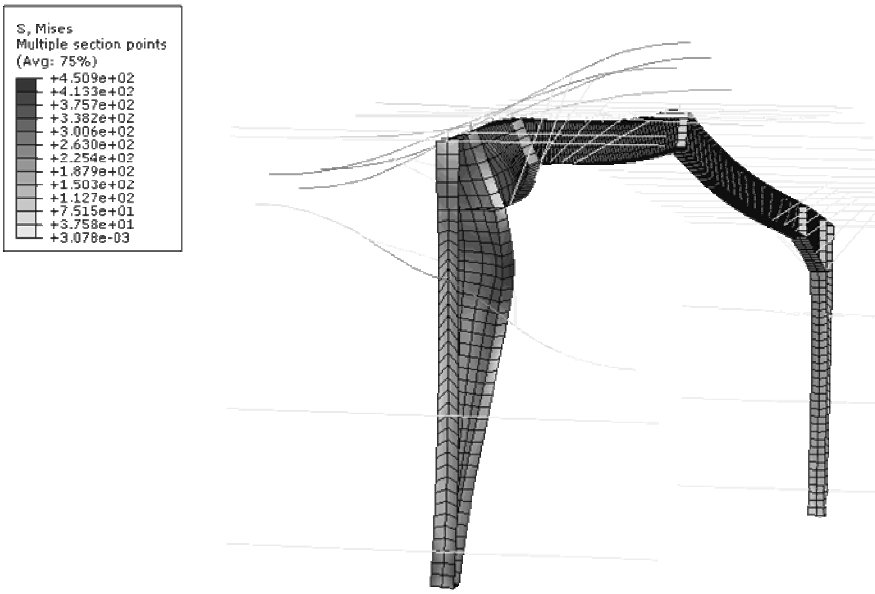

Figure 11. The lateral torsional buckling failure of 6 m high frame with 32 m span (5x scaled) considering support of purlins.

The calculation covers 6 m high frames with basic snow load 2500 N/m^2 and basic wind velocity 30 m/s according to EN 1991 [19]. The weight of welded-tapered frames without lateral supports as well as fully restrained frames is optimized for comparison (Figure 12). As it can be observed, the lateral supports in smaller frames restrains the out-of-plane buckling very well while the effect of those supports rapidly decreases with increasing span. This indicates that light gauge steel purlins can be relied upon for stabilizing smaller span frames, but they are not effective when the span increases.

The graphs can also give an indication to the designer whether providing diagonal stays is economical. Implementing diagonal stays in the design of structure brings an extra expense: it requires connecting plates to be welded on the main frame and it very often disrupts the internal sheeting. The gain in terms of material consumption on the frame might not offset the expenses. In any case, the lateral stabilization of the frame has to be modelled using 3D model with proper support stiffness to correctly account for their effect.

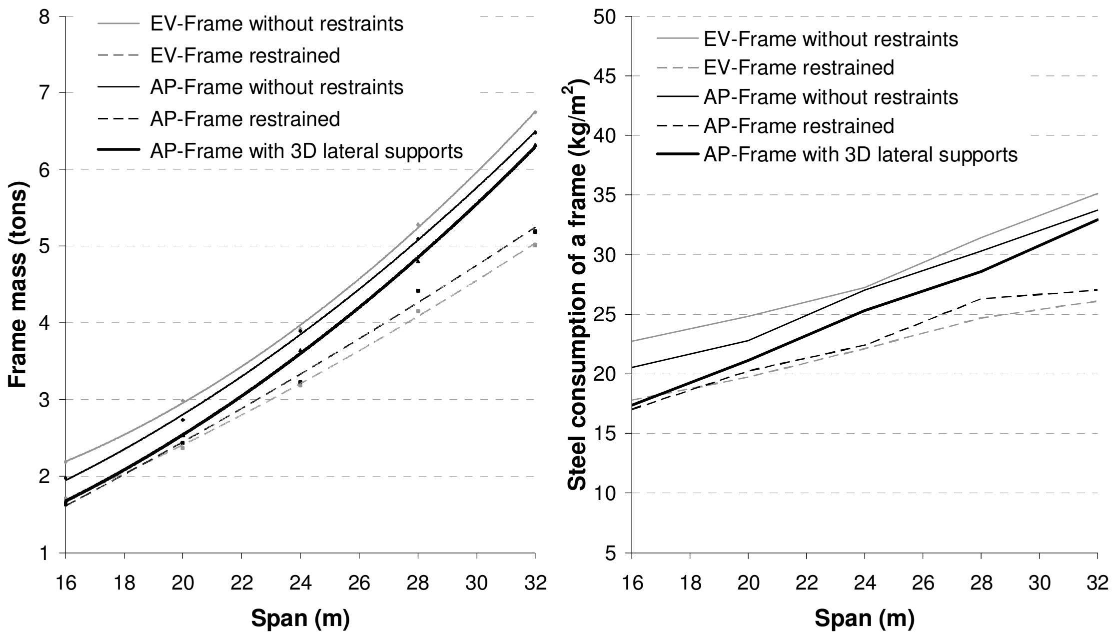

Figure 12. The effect of out-of-plane buckling studied on 6 m high welded-tapered frames with purlins and diagonal stays included in the model (solid fat line) compared to models without any lateral restraints (solid thin line) and fully restrained (dashed line) using different design methods.

Conclusions

The advanced 3D modelling methods explored in this paper are very effective, and offer the designer the ability to model complex structural configurations and support conditions. Because the simplified methods described in the design codes for special configurations are naturally conservative, the use of advanced options can bring substantial economical benefits.

On the other hand, due to advances in computing capabilities (both software and hardware), the use of these 3D shell models is not any more out of reach of an average design office. The calculation time in the range of a few minutes with the AP-frame tool is still long compared to the runtime in range of few seconds with beam-based analysis software, but both of them are much shorter compared to the time spent on creating the model. If the model preparation is automated (like in case of AP-Frame) the advanced tool is very competitive for single analysis.

In terms of optimization, the use of EV-Frame offers instant solution. It is clear that similar tools should be, and for some cases are, used by the design offices. The GA offers a strong and versatile option for optimization in the field of structural engineering.

The optimization time of advanced modelling AP-Frame tool is still in the range of many hours when used on a standard computer. However, the combination of 3D modelling and GAs can take the advantage of parallel computing and is especially suitable for server applications.

Concerning the specific results presented in this paper, it has been shown that using slender welded-tapered frames instead of hot-rolled sections leads to decreased steel consumption. Cost savings could be achieved by implementing modern fabrication technologies. However, slender frames have to be properly designed especially considering their lateral stability. In order to effectively design the lateral restraints, calculation of 3D model is needed. The usual diagonal stay configuration of lateral restraint, when it relies on light-gauge steel purlins, is not effective in preventing the lateral buckling of the frame, especially in larger span frames. Further, implementing shape optimization into the design process provides economical solution tailored to the specific loading situation.

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Petr Hradil, Matti Mielonen, Ludovic Fülöp
VTT Technical Research Centre of Finland
P.O.Box 1000
FIN-02044 VTT
FINLAND
petr.hradil@vtt.fi, matti.mielonen@vtt.fi, ludovic.fulop@vtt.fi