

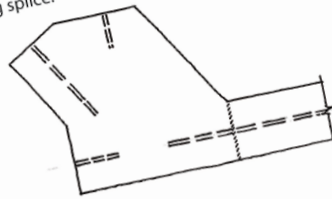
NSC

TECHNICAL DIGEST 2019

them. The webs would need to be... the joint zone. This arrangement is not favoured. Rotating the members so their flanges are vertical (Figure 4) provides a more direct path for the flange tension forces.

The connections between the members in the node can be made by butt welds smoother but the web force still needs to be transferred, and the junction where the webs of the three members come together is complicated. A refinement of this arrangement, using two plates to form the node, separated by intermittent webs is the favoured solution (Figure 5). The plates toward the centre of the joint are wide enough to carry half the chord force so a web is only required close to the connecting member to transfer the web force into the plates. The plates are butt welded to the chord flanges and the tension diagonal is connected using a bolted splice. The column member is connected using a bearing splice.

Figure 5: Bottom chord joint – splice plates



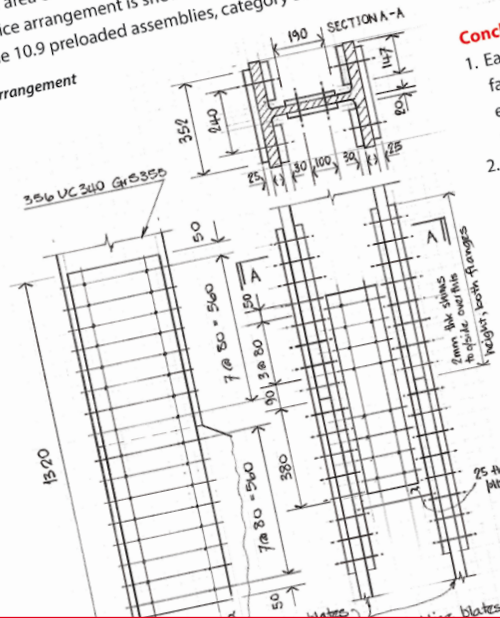
Joint design

The tension splice in the bracing member will be effected using M30 preloaded bolts of category B in double shear, of grade 10.9. The slip resistance assumed for design is for a friction coefficient $\mu = 0.5$ and is 357 kN. The member is a 356 UC 340 with 42.9 mm thick flanges and 26.6 mm thick web and has an area of 433 cm². The area of one flange is 40% of the total and carries 5.2 MN in tension. The number of bolts required is indicated in Table 2:

	Force (MN)	No of Bolts	Adopt
Flange	5.2	14.6	16 bolts
Web	2.6	7.3	8 bolts

The flange splice plates are chosen to provide the same area of metal as the flange with half the area on each side to balance the force on each shear plane in a bolt. The splice arrangement is shown in Figure 6. All bolts in the truss will be M30 grade 10.9 preloaded assemblies, category B.

Figure 6: Bolted splice arrangement

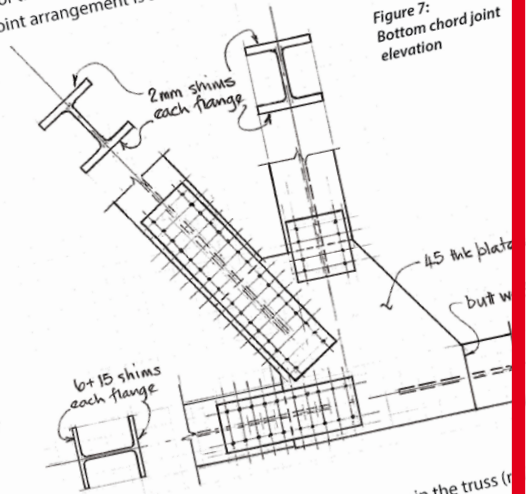


of the maximum... resistance gives 7.6 bolts and four... The tension connection for the continuation of... detailed in a similar way to the tension diagonal with ten bolts in... and six bolts in the web. The difference in flange thickness in this part of the joint is 21 mm and is achieved with two shims of 15 mm and 6 mm thickness.

The connection between the bottom chord and the node plates is required to transfer 14.2 MN in tension. The node plates will be butt welded to the bottom chord member. The force in the web will be transferred by welds to the node plates; either fillet welds or partial penetration butt welds can be used. This can be achieved either by stripping both flanges off the member to allow the web to project between the node plates or by butt welding an extension plate to the web. Short web plates are required between the node plates at each bolted connection to receive the web force and transfer it into the node plates through fillet welds.

The node plate geometry is such that the stress in the plates reduces rapidly away from the interface with the connected members. The resistance to compression from the vertical column must also be considered. A buckling check of the compression force in the node plates should be carried out. The final joint arrangement is shown in Figure 7.

Figure 7: Bottom chord joint elevation



Conclusions

1. Early consideration of the form of members in the truss (fabricated) may influence the depth adopted. Transportation and erection and the proportion of shop fabrication also influence the arrangement.
2. The flow of forces through the joint is clear if the force for and nature of welds required to transfer the force and orientation of members (webs vertical or horizontal) is considered.
3. Facilitating the flow of forces between the element arrangement where member stubs are not welded provided, aligned with the stiffening required and the joint which reduces the stiffening required and the force.
4. Fewer bolts would result if non preloaded grade 10.9 bolts (30 on each side of the diagonal member splice) were used. Deflection of the truss would be more difficult to control and movement in clearance holes and bearing defects would be more likely.
5. The double shear resistance of non preloaded bolts is 4% greater than grade 8.8 bolts of the same size. This would produce a smaller number of bolts. Once selected, the bolts are fixed for the whole truss.

Reference

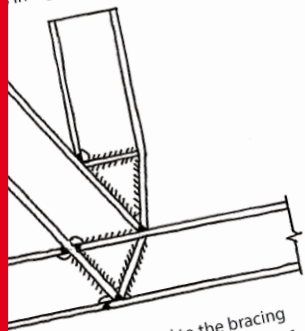
1. Steel construction with trusses, NSC, March 2017.

017'.
choices

arrangement
Figure 2.



members
column carrying 10 MN.
the fabricated assembly
diagonal brace and the
tension in the central
tension diagonal carrying
40% of the tension in each
orientation of the members
in Figure 3.



forces from the chord to the bracing
forces are obviously too large to transfer
load bearing stiffener is necessary
change in direction. As the forces are in
direction, the butt welds between the
are cope holes through the web to achieve



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Nick Barrett - Editor

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Illustration of fatigue design of a crane runway beam

As indicated in the technical article^[1] in the September 2018 issue of *New Steel Construction* Richard Henderson of the SCI discusses the fatigue design of crane runway beams with an illustrative design example.

Crane Loading

The loads on crane runway beams are determined in accordance with BS EN 1991-3^[2]. This code sets out the groups of loads and dynamic factors to be considered as a single characteristic crane action. The relevant partial factors are set out in Table A.1 in Annex A of the code. At ultimate limit state for the design of the crane and its supporting structures, the characteristic crane action being considered is combined with simultaneously occurring actions (eg wind load) in accordance with BS EN 1990. The final ultimate design loads from the crane end carriage which are supported by the runway beam can thus be determined.

The groups of loads are identified in Table 2.2 of BS EN 1991-3 and include the actions listed in the table below. Several of the loads have a dynamic factor associated with them which depend on the class and function of the crane.

Item	Description of load	Dynamic factor
1	Self-weight of crane	φ_1 OR φ_4
2	Hoist load	φ_2, φ_3 OR φ_4
3	Acceleration of crane bridge	φ_5
4	Skewing of crane bridge	-
5	Acceleration or braking of crab or hoist block	-
6	In-service wind	-
7	Test load	φ_6
8	Buffer force	φ_7
9	Tilting force	-

Unfavourable crane actions have a γ_Q value of 1.35, not the usual value of 1.5. Fatigue assessment is regarded as a serviceability limit state with a partial factor of 1.0.

Fatigue Assessment

BS EN 1991-3 provides a simplified approach to designing crane runway beams (gantry girders) for fatigue loads to comply with incomplete information during the design stage, when full details of the crane may not be available. The crane fatigue loads are given in terms of fatigue damage equivalent loads Q_e that are taken as constant for all crane positions. The fatigue load may be specified as follows:

$$Q_e = \varphi_{fat} \lambda_i Q_{max,i}$$

where, as stated by the code, $Q_{max,i}$ is the maximum value of the characteristic vertical wheel load, i and $\lambda_i = \lambda_{1,i} \lambda_{2,i}$ is the damage equivalent factor to make allowance for the relevant standardized fatigue load spectrum and absolute number of load cycles in relation to $N = 2.0 \times 10^6$ cycles. This concept was discussed in reference [1].

The damage equivalent dynamic impact factor φ_{fat} for normal conditions may be taken as:

$$\varphi_{fat,1} = \frac{1 + \varphi_1}{2} \text{ and } \varphi_{fat,2} = \frac{1 + \varphi_2}{2}$$

The factors $\varphi_{fat,1}$ and $\varphi_{fat,2}$ apply to the self-weight of the crane and the hoist load respectively.

In BS EN 1991-3, Annex B Table B.1 gives recommendations for loading classes S in accordance with the type of crane and Table 2.12 gives a single value of λ for each of normal and shear stresses according to the crane classification. Overhead travelling cranes are in either S-class S6 or S7 so that, having selected an S class, the corresponding λ value is determined. (The classes S_i correspond to a stress history parameter s defined in BS EN 13001-1^[3] but the details are not required for this example).

The method for carrying out the fatigue assessment is set out in section 9 of BS EN 1993-6^[4]. Once the fatigue loads are determined, the stress ranges (denoted $\Delta\sigma_{E,2}$) for the critical details of the crane runway beam can be calculated. These are the damage equivalent stress ranges related to 2 million cycles. The fatigue stress range is multiplied by the partial factor for fatigue loads γ_{ff} stated in BS EN 1993-6 section 9.2 which is equal to 1.0. The critical details must be categorized according to Tables 8.1 to 8.10 in BS EN 1993-1-9 and the detail category number noted. The category number (denoted $\Delta\sigma_c$) is the reference value of the fatigue strength at 2 million cycles. The partial factor for fatigue strength is γ_{Mf} and is given as 1.1 in the National Annex to BS EN 1993-1-9 for a safe-life fatigue assessment. The fatigue check involves showing that, for direct stresses:

$$\frac{\gamma_{ff} \Delta\sigma_{E,2}}{\Delta\sigma_c / \gamma_{Mf}} \leq 1.0$$

A similar check is required for fluctuating shear stresses:

$$\frac{\gamma_{ff} \Delta\tau_{E,2}}{\Delta\tau_c / \gamma_{Mf}} \leq 1.0$$

If both direct and shear stresses are present, a further check is required.

Example

Consider an EO travelling crane of S-class 6 and hoisting class HC3 supported on 8.0m span runway beams in steel grade S355 which have laterally restrained compression flanges at 2.0 m centres. The crane is wholly inside a building and so there are no other simultaneously occurring actions. The relevant weights of the crane, the proportion of the weight applied to the end carriage in the worst case and the resulting maximum loads are:

Item	Load (kN)	Proportion of load	Load on end carriage (kN)
End carriage and bridge (Q_c)	164	50%	82
Crab (Q_c)	36	90%	33
Payload (Q_h)	300	90%	270

For the purpose of this example, consider load group 1 from Table 2.2 of BS EN 1991-3:

$$\varphi_1 Q_c + \varphi_2 Q_h + \varphi_3 (H_L + H_r)$$

where H_L and H_r are caused by acceleration or deceleration of the crane bridge and for simplicity will not be considered further. From Table 2.4 of BS EN 1991-3, the upper-bound value of $\varphi_1 = 1.1$ and the value of φ_2 is given by:

$$\varphi_2 = \varphi_{(2,min)} + \beta_2 v_h$$

Figure 1: Ultimate bending moments

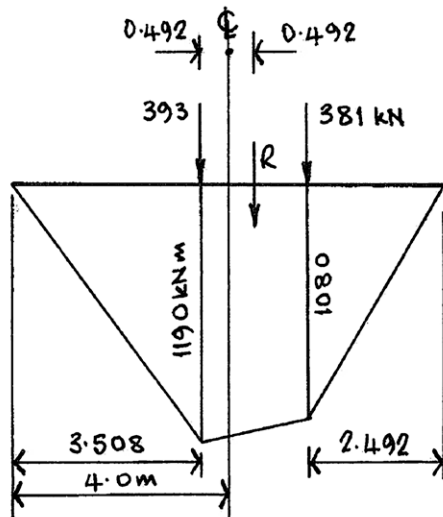
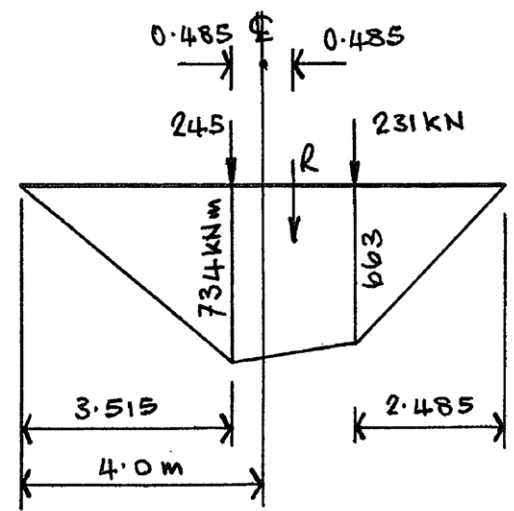


Figure 2: Bending moments from fatigue loads



where v_h is the steady hoisting speed and β_2 is a coefficient. According to Table 2.5 of BS EN 1991-3, for hoisting class HC3, $\varphi_{2,min} = 1.15$ and $\beta_2 = 0.51$. Taking the steady hoisting speed as $v_h = 1.0 \text{ ms}^{-1}$, the value of φ_2 is 1.66. Applying the dynamic factors gives the following loads:

Item	Dynamic factor	Factored Load on end carriage (kN)
End carriage and bridge (Q_c)	1.1	90
Crab (Q_c)	1.1	36
Payload (Q_h)	1.66	448

The crane end carriage will be assumed to have wheels 2.0 m apart and the loads are distributed between them as indicated in the table below (the weight of the crane bridge is assumed not to be distributed evenly). The ultimate loads on each wheel are as indicated:

Item	Load Wheel 1 (kN)	Load Wheel 2 (kN)	Total (kN)
End carriage and bridge (Q_c)	50	40	90
Crab (Q_c)	18	18	36
Payload (Q_h)	224	224	448
Ultimate load (factor = 1.35)	393	381	774

The maximum moment in the beam occurs when the centre of the span bisects the distance between the resultant of the loads and a wheel load as shown in figure 1.

The maximum bending moment is 1190 kNm. Assuming a uniform bending moment between compression flange restraints, using the Blue Book, a $610 \times 229 \text{ UB } 125$ with restraints at 2.0 m centres has a buckling resistance moment (with $C_1 = 1.0$) of 1230 kNm which is satisfactory for ultimate loads. The elastic modulus of the beam W_e is 3220 cm^3 .

As indicated above, BS EN 1991-3 gives a simplified approach to calculating the fatigue damage equivalent load Q_e which may be expressed as follows:

$$Q_e = \varphi_{fat} \lambda Q_{max,j} = \lambda [\varphi_{fat,1} (Q_{max,i})_1 + \varphi_{fat,2} (Q_{max,i})_2]$$

where $\varphi_{fat,j} = (1 + \varphi_j)/2$ and the index j refers to the dynamic factor. Substituting values for φ_1 and φ_2 gives $\varphi_{fat,1} = 1.05$ and $\varphi_{fat,2} = 1.33$. and calculating the characteristic and fatigue damage equivalent loads gives the following results:

Item	Load Wheel 1 (kN)	Load Wheel 2 (kN)	Total (kN)
Characteristic load ($Q_{max,i}{}_1$)	62	53	115
Characteristic payload ($Q_{max,i}{}_2$)	135	135	270
$\Sigma \varphi_{fat,j} (Q_{max,i}{}_j)$	245	231	

The maximum bending moment in the beam is shown in Figure 2 and is

equal to 734 kNm.

The maximum direct stress due to fatigue loads is therefore 228 MPa. The self-weight bending moment at the same position is about 9.7 kNm which gives a stress of about 3.0 MPa. Table 2.12 of BS EN 1991-3 gives a single value of $\lambda = 0.794$ for direct stress for class S_6 .

The fatigue stress range is therefore:

$$\Delta\sigma_{e,z} = (228 \times 0.794) - 3.0 = 178 \text{ MPa}$$

Consider the bottom flange first: the detail category is 160 which corresponds to a rolled section with as-rolled edges, fettled in accordance with the requirements stated in BS EN 1993-1-9 Table 8.1 for the relevant detail category, so $\Delta\sigma_c = 160 \text{ MPa}$. For the fatigue verification, considering direct stress:

$$\frac{\gamma_{FF} \Delta\sigma_{e,z}}{\Delta\sigma_c / \gamma_{MF}} \leq 1.0$$

so, substituting values:

$$\frac{1.0 \times 178}{160 / 1.1} = 1.23 \text{ — fails!}$$

The fatigue load case is obviously more critical than the ultimate load case. Note that the highest fatigue class was chosen for the assessment. If the top flange is considered and the crane rail is fastened to the top flange with bolted cleats (a more onerous case), the relevant detail category is 90 (description: structural element with holes subject to bending and axial forces) and the factored fatigue stress is about 82 MPa. The stress $\Delta\sigma_{e,z}$ must be less than this value to satisfy the verification equation so a much larger beam is required. The elastic modulus must at least equal:

$$3220 \times \frac{178}{82} = 6990 \text{ cm}^3$$

A $914 \times 305 \text{ UB } 201$ has an elastic modulus of 7200 cm^3 . This beam has a buckling resistance moment of 1310 kNm for a length of 8 m between lateral restraints so no intermediate restraints are required.

For a complete assessment, the axial and transverse forces which have been neglected increase the stresses in the beam and must be considered.

References

- [1] Henderson R, Introduction to fatigue design to BS EN 1993-1-9, NSC, September 2018
- [2] BS EN 1991-3: 2006 Eurocode 1 – Actions on structures Part 3: Actions induced by cranes and machinery
- [3] BS EN 13001-1:2015 Cranes – General Design Part 1: General principles and requirements
- [4] BS EN 1993-6: 2007 Eurocode 3 Design of steel structures – Part 6: Crane supporting structures
- [5] BS EN 1993-1-9:2005 Eurocode 3 Design of steel structures – Part 1-9 Fatigue

Properties of “Z grade” steel

David Brown of the SCI discusses the specification of steel with improved through thickness properties. It should be noted that steel with through thickness properties (so-called “Z grade”) is only needed in high risk situations.

Steel with improved through thickness properties is often referred to as “Z grade”, although the formal description is ‘Quality class’. The “Z” is simply because the dimensions in-plane are “x” and “y” and out-of-plane, through the thickness of the material, is the “z” direction. The word “improved” is important, as steels to the EN 10025 Standards will generally have resistance to stress in the z direction. The common arrangement used to demonstrate the potential need for improved through thickness properties is shown in Figure 1 – tensile stress is applied through the ‘incoming’ plates, leading to possible lamellar tearing in the ‘through’ plate. Lamellar tearing is when the steel in the ‘through’ plate separates internally.

Internal tearing may occur due to areas of inclusions or impurity which can be detected by ultrasonic testing, or when through thickness loading causes tearing to propagate between micro imperfections. Micro imperfections cannot readily be detected by ultrasonic testing, but would be revealed by through thickness testing to EN 10164.

Material specification

Steel may be examined for the two types of imperfections mentioned above by specifying certain options at the time of order. Within EN 10025, which covers the steel sections and plate normally used in construction, options 6 and 7 apply to plate and sections with parallel flanges respectively, and require the steel to be examined for internal defects by ultrasonic testing. If through thickness properties are required, this must be selected by specifying option 4, which is testing in accordance with EN 10164. If through thickness testing to EN 10164 is specified, this automatically includes ultrasonic testing to EN 10160 (for plate) or EN 10306 (for sections) as applicable, so there is no need to separately specify option 6 or 7.

Through thickness testing

Through thickness testing to EN 10164 requires samples cut from the plate (or section) to be subject to a tensile force in the z direction until the sample fractures. The test is examining the capacity of the steel to ‘neck’ before fracture, which is a measure of material ductility in the z-axis. The samples

are machined to have a circular cross section, typically of 6 mm or 10 mm diameter, with a “headed” portion of the form shown in Figure 2, so that it can be gripped in a testing machine. EN 10164 specifies where the samples are to be taken – typically at 1/3 of the web depth and 1/3 of the flange outstand (measured from the tip).

The obvious question relates to the testing of thin material – how can this be prepared in such a way to be gripped in a testing machine? For thin material, extension pieces are welded to the sample. Because welding will change the material properties locally, the original sample must be at least 15 mm thick. To minimise the effect of the welding, EN 10164 suggests that extension pieces be friction welded to ensure the heat affected zone is minimised. Fracture in the weld or heat affected zone invalidates the results. Extension pieces are mandatory for samples up to 20 mm thick, optional for samples between 20 and 80 mm thick, and cannot be used for samples thicker than 80 mm.

Three samples are tested and in each case the reduction of area when the sample fractures is given by:

$$\frac{S_o - S_u}{S_o} \times 100$$

where S_o is the original cross sectional area,
 S_u is the minimum cross sectional area after fracture.

Both the average and individual results are needed to define the quality class in accordance with Table 1.

Quality class	Reduction of area in %	
	Minimum average value of three tests	Minimum individual value
Z15	15	10
Z25	25	15
Z35	35	25

Table 1: Z Quality class

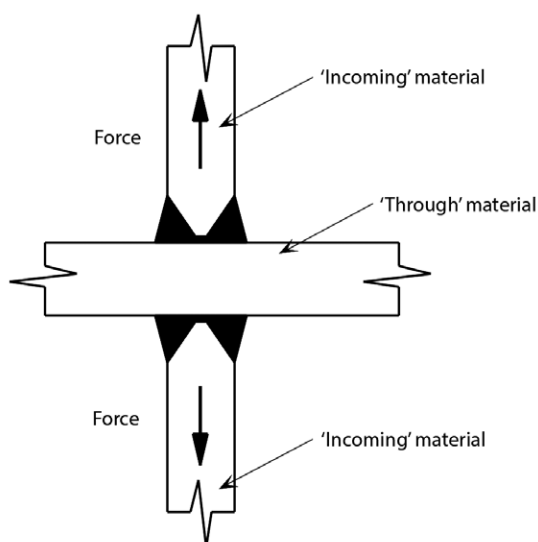


Figure 1 – Cruciform joint

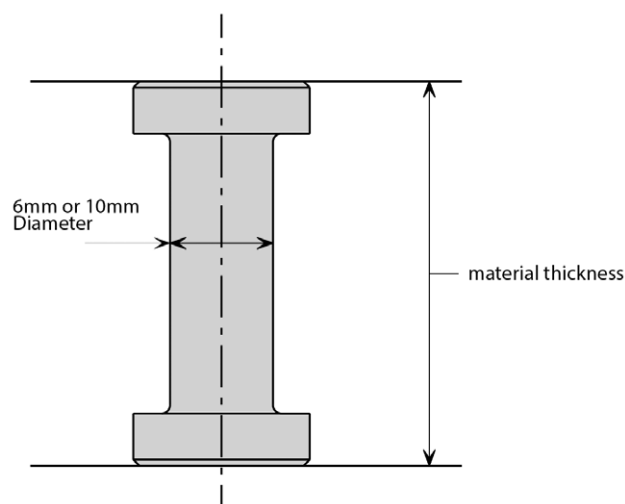
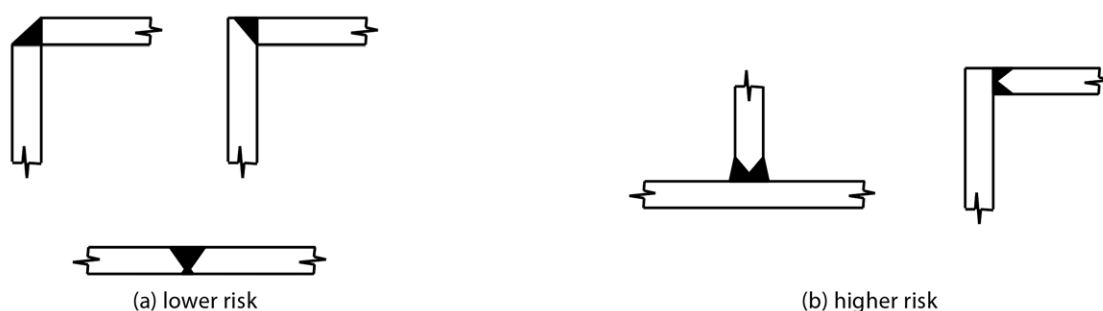


Figure 2 – Testing sample profile

Figure 3: Joint types



Eurocode requirements

A procedure to determine if improved through thickness properties are required is given in Section 3 of BS EN 1993-1-10. Readers should note that there is little enthusiasm in the UK for this procedure, and alternative guidance is given in PD 6695-1-10. Despite the UK position, the guidance in BS EN 1993-1-10 establishes important principles, reinforced by the PD. The Eurocode notes that:

- The strain through the thickness of the material arises as welds to the surface (see Figure 1) cool and shrink. If that shrinkage is restrained by other stiff parts of the assembly, it is clear that the possibility of lamellar tearing increases,
- Larger welds increase the possibility of tearing,
- Thoughtful weld detailing can reduce the risk, for example by avoiding fusion faces which are parallel to the surface of the steel,
- The sulphur content in the steel is important – lower levels improve the through thickness properties of the steel.

The procedure in BS EN 1993-1-10 is essentially a scoring system based on a number of contributing factors. Criteria that increase the risk are awarded a higher score, those that reduce the risk given a lower or negative score. The required Z quality class (Table 1) must be greater than the summation of the individual scores. Some examples illustrate the features of the system:

A fillet weld throat 5 mm scores zero, a throat of 14 mm scores 6. The table includes fillet welds up to a 35 mm throat with a score of 15, but would be unusual, one hopes!

Welds where the fusion faces are not parallel to the surface (Figure 3a) score -25 (indicating that these are not a problem). Welds made to the surface of the steel (Figure 3b) score 5, or 8, depending on the detail.

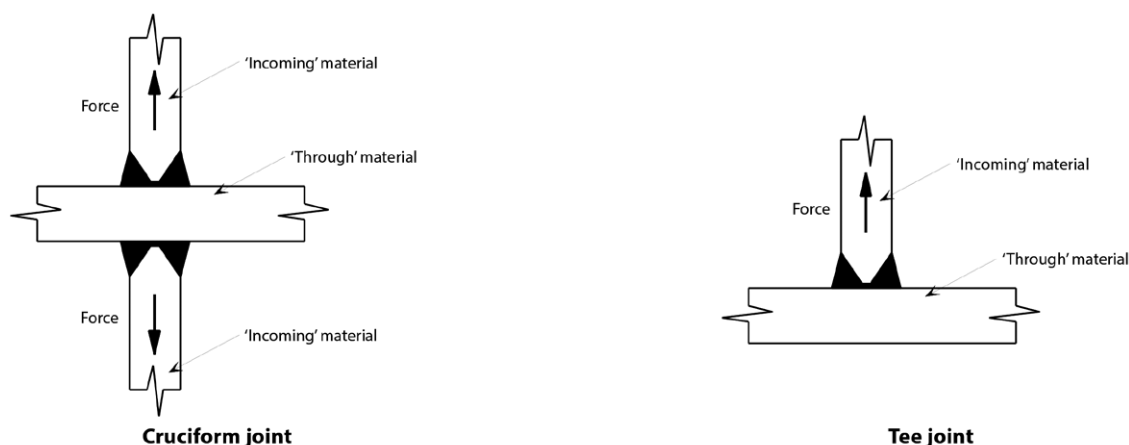
Thicker material, which provides more restraint, scores between 2 for 10 mm material and 15 for 70 mm material.

Perhaps surprisingly, the degree of restraint offered by other portions of the assembly is not so significant – a score of zero for low restraint to (a mere) 5 for high restraint. The most significant contributions are therefore the weld size, the thickness of the material and the joint type.

Guidance in PD 6695-1-10

The UK guidance is that through thickness testing is expensive, often unnecessary, and should only be specified in 'high-risk' situations. High-risk situations, illustrated in Figure 4, are identified as:

Figure 4: 'High risk' situations



- Tee joints with butt welds where the thickness of the 'incoming' material is greater than 35 mm, or if fillet welded the throat is greater than 35 mm (again, a notable fillet weld!)
- Cruciform joints with butt welds where the thickness of the 'incoming' material is greater than 25 mm, or if fillet welded the throat is greater than 25 mm (still notable!)

In these high risk situations, the specification of quality class Z35 is recommended. If Z35 material cannot be readily obtained, then the sulphur content should be limited to 0.005%. This is significantly lower than the maximum specified in BS EN 10025-2, which is typically 0.03%.

In addition, weld volume should be minimised by avoiding over-specification – which is sensible advice in all situations. Both the designer and steelwork contractor can contribute here: the designer by not specifying conservative forces for the connection design and the steelwork contractor by making a careful choice of joint preparation.

PD 6695-1-10 notes that steel with low sulphur levels is likely to have improved through thickness properties (Z25 or even Z35) as a matter of course. The sulphur levels which have such a significant influence on through thickness properties may be verified by looking at the mill certificates. The PD also lists a series of practical measures to reduce the risk of lamellar tearing. These measures are primarily for the steelwork contractor and reflect the contributions to the overall risk score noted above. Practice to reduce the risk includes:

- Avoiding weld details where the fusion face is on the surface of the material.
- Managing the assembly of fabricated items to reduce restraint on subsequent welds.
- Minimising shrinkage of the welds by process control.
- Ordering steel with lower maximum sulphur levels, or purchasing steel from suppliers known to produce 'cleaner' steel.

Conclusions

In Western and other developed countries, steel is likely to be 'clean' (low sulphur), the steelwork contractors undertaking complex welding of large assemblies are likely to be highly experienced and the welding operations will be managed by a Responsible Welding Coordinator (an essential individual for the production of CE Marked steelwork). In these circumstances improved through thickness properties need only be specified for the high risk situations noted above.

Stability and second order effects on steel structures: Part 1: fundamental behaviour

Ricardo Pimentel of the SCI introduces the topics of buckling phenomenon, second order effects and the approximate methods to allow for those effects. In part 2, the various methods will be compared to the results from a rigorous numerical analysis.

When a structure is loaded, deformation occurs, and the internal forces within the structure are modified. If at some point an increase of load (and deflection) does not modify the internal forces, the structure becomes unstable (only considering elastic buckling). In a perfect structure, a theoretical sudden instability exists when the applied loads reach a critical load. However, because real structures are always imperfect, the so-called sudden instability does not exist – an initial bow imperfection in a strut will increase as the applied load increases. When the applied load becomes closer to the theoretical critical value, the deformation increases rapidly. This leads to the following conclusions: (i) when loaded, a strut tends to diverge from its initial position “guided” by the initial bow imperfection; (ii) the magnitude of the initial bow imperfection will have influence in the critical load of the strut; (iii) the applied load will have impact on the deformed shape, which in turn will influence the buckling resistance of the member.

From the concepts explained above, the assessment of instability problems must consider the effects of the deformations due to the applied loads. Even for the theoretically perfect structures, the prediction of the load that leads to sudden instability requires the assumption of a deformed shape of the system. To address the problem, taking the frame in Figure 1 as example, two types of effects are important:

- (i) $P-\delta$ effects, which are related to deformations within the length of members, and
- (ii) $P-\Delta$ effects, which are related to movement of nodes.

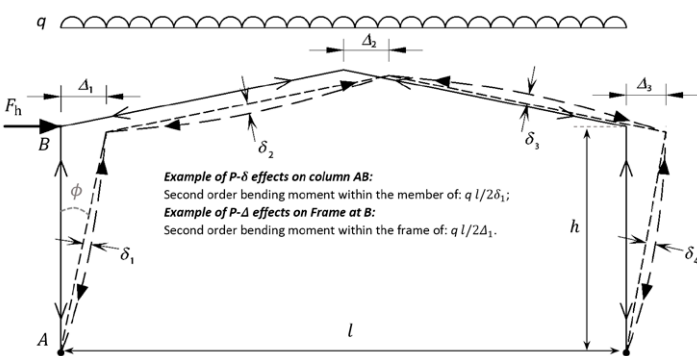


Figure 1 – Local (δ) and global (Δ) displacements which produce second order effects $P-\delta$ and $P-\Delta$.

The impact of the $P-\delta$ and $P-\Delta$ effects is to change the forces and deflections within the structure. These are second order effects, not accounted for in a usual first order analysis. Second order effects may be accounted for by a geometric non-linear analysis or by approximate modifications of a first order analysis. A second order analysis can be done through a series of first order analyses, applying the load in small increments, but for each increment, the deformed shape of the structure is considered.

For an idealized “perfect” pin-ended strut (Figure 2), the theoretical

critical load that leads to a sudden instability of the system can be obtained by solving a second order differential equation¹. In the process, the displacement “ y ” along “ z ” is established using a sinusoidal function, which later leads to the following definition:

$$P = \frac{n^2 \pi^2 EI}{l^2} \text{ where } n=1,2,3\dots$$

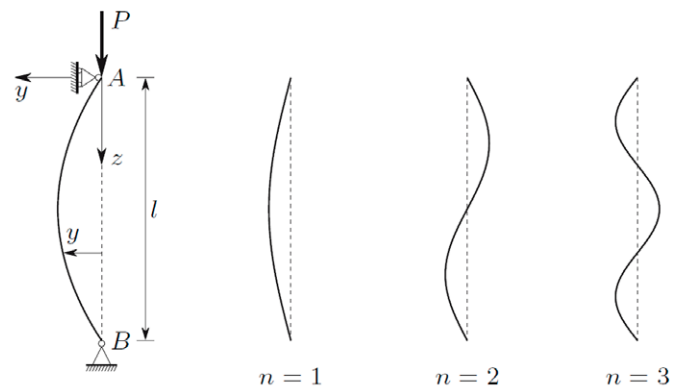


Figure 2 – Buckling modes for a pin-ended strut².

The load P is the Euler buckling load. It is clear that there are many possible values for P with different value of “ n ” leading to different buckling mode shapes. These modes are usually called eigenvalues. The minimum value of P ($n=1$), represents the critical load of the strut (P_{cr}), which means that the first eigenvalue of the system will represent the critical buckling mode shape.

The governing equation can be re-arranged for different boundary conditions as presented in Figure 3. For some configurations (such as “a”, “b” or “c”), with geometric/symmetric considerations a solution is possible without solving the differential equation. For example “a”, it is clear that the critical configuration has the same shape of a pin-ended member with an equivalent length of $2l$. The corresponding critical load for case “a” is presented in the expression below ($P_{cr,a}$). The term l_{eff} is the so-called effective length, which may be defined as the length that a pin-ended strut with the same cross-section that has the same Euler load as the member under consideration.

$$P_{cr,a} = \frac{n^2 \pi^2 EI}{2l^2} \text{ or } = \frac{n^2 \pi^2 EI}{l_{eff}^2}, \text{ therefore } l_{eff} = 2l$$

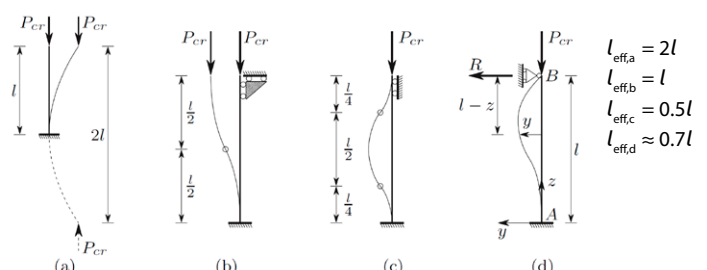
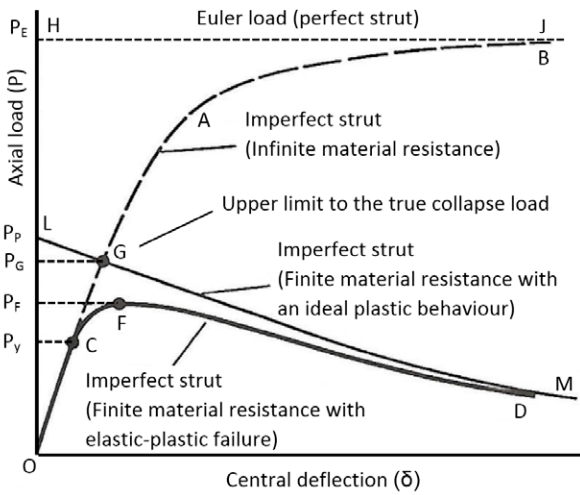


Figure 3 – Effective length for struts with different boundary conditions².



Notes: for an imperfect strut with finite material resistance (curve OCFD), after reaching yield (Point C), there is a clear decrease of stiffness due to plasticity, making the behaviour diverge from the elastic response (line OCG).

- P – Axial Load;
- P_E – Euler Load;
- P_Y – Load to elastic resistance;
- P_F – Load in failure with elastic-plastic behaviour;
- P_P – Load to ideal plastic resistance (squash load);
- P_G – Load in failure with a perfect plastic hinge;
- σ_y – Yield strength of the material.

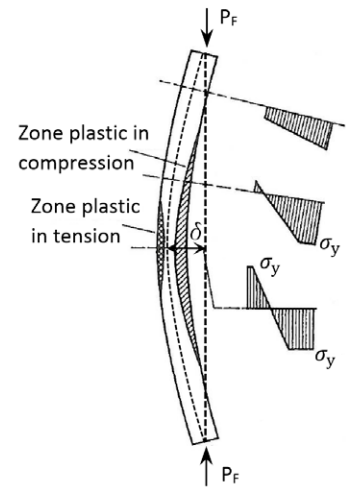
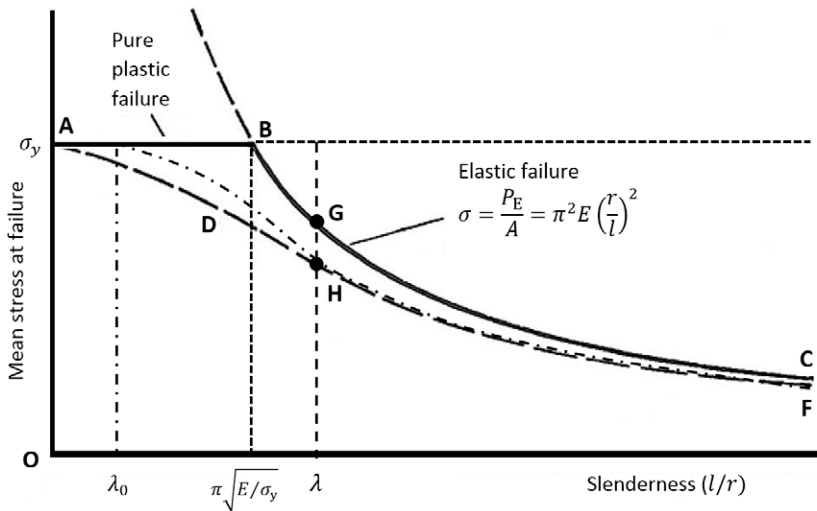


Figure 4 – Response of a strut under axial load^{5,6}



- P_E – Euler Load;
- σ_y – Yield strength of the material;
- σ – Allowable stress;
- l – Strut length;
- r – Radius of gyration;
- λ – Slenderness;
- E – Young modulus;
- A – Section Area.

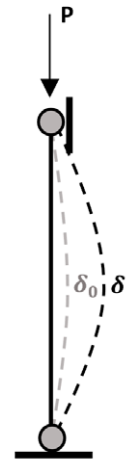


Figure 5 – Response of a real strut under axial load⁵

The behaviour presented below left represents a “perfect” strut. However, imperfections will always exist, creating additional flexure in the element. This will limit the resistance to loads lower than the Euler load (line HJ in Figure 4). The residual stresses due to manufacture processes will also contribute to a lower resistance. Eurocode 3 deals with initial imperfections by specifying an equivalent bow imperfection which allows for all these effects. The behaviour of a real strut can be represented by line OCFD in Figure 4, where it is clear that the maximum axial resistance is between the elastic (Point C) and the plastic resistance of the cross section (Point G). As the resistance of Point F is difficult to determine, the calculated resistance is conservatively taken as Point C. According to clause NA.2.11 of the UK NA to EN 1993-1-1³, to obtain the initial bow imperfection, the designer should complete a back-calculation using the buckling design procedure according to EN 1993-1-1⁴ section 6.3. For the reasons explained, the elastic section modulus should be used in the process.

Figure 5 shows the Euler buckling curve (presented as stresses) which is an upper limit to the resistance. AB represents the plateau where according to theory, there is no buckling. At slenderness λ , Point G would represent the theoretical resistance, but this is reduced to Point H, due to the effect of local imperfections.

The Eurocode introduces an initial plateau (limited by λ_0 in Figure 5) for the design of imperfect struts. According to clause 6.3.1.3 of EN 1993-1-1, the plateau is determined by $\bar{\lambda} = 0.20$, where $\bar{\lambda} = \sqrt{A\sigma_y/P_{cr}}$ (the Eurocode terms are $\bar{\lambda} = \sqrt{Af_y/N_{cr}}$). This plateau makes an allowance for strain

hardening in short columns⁶. For values above the specified slenderness for the plateau, second-order $P-\delta$ effects are always relevant for members.

The differential equation for the “perfect” struts in Figure 2 can be adapted to consider an initial bow imperfection. If the formulation for a “perfect” problem is rather complex, including an initial imperfection would certainly be more so. However, to demonstrate the concept of the effects of an initial bow imperfection, a simplified model can be adopted, where the system from Figure 2 is replaced by an idealized problem having a joint with a spring stiffness as shown in Figure 6^{2,6}.

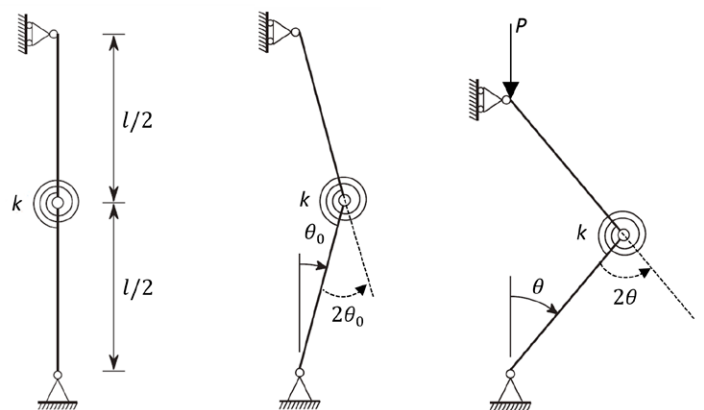


Figure 6 – Idealized system with a joint with a spring stiffness².

Assuming that the upper and lower bars have an initial rotation " θ_0 ", with zero rotation of the spring, and an axial load is applied, the rotation increases to θ , and the moment on the spring becomes $M_{spring} = k \cdot 2(\theta - \theta_0)$, where k is the (elastic) spring stiffness. The equilibrium in the deformed shape leads to the following expression: $P\theta l / 2 = M_{spring}$. From the two previous expressions, it can be shown that $P = \frac{4k}{l} \left(\frac{\theta - \theta_0}{\theta} \right)$. The critical buckling load P_{cr} is for a perfectly straight member, i.e. $\theta_0 = 0$. In this case, $P_{cr} = 4k/l$.

Therefore, $P = P_{cr} \left(\frac{\theta - \theta_0}{\theta} \right)$. If $\theta_0 \neq 0$, θ would need to be infinite for P to be equal to P_{cr} . This means that the imperfect column will never reach the Euler load (this is consistent with the line OCGAB from Figure 4). The equation can be re-written as $\theta = \left(\frac{1}{1-\mu} \right) \theta_0$, where $\mu = P_{cr} / P$. This is the so-called

amplification factor. This factor allows the consideration of second order effects by amplifying the first order effects. EN 1993-1-1 section 5.2.2 introduces this factor for frame stability in the form of $\frac{1}{1-\alpha_{cr}}$ which leads to

$\alpha_{cr} = P_{cr} / P$, where P is the applied load and P_{cr} is the elastic critical load (for a strut, this will be Euler load). From a rigorous calculation, it can be justified that the simplified formulation provides reasonable results for $P \leq 0.5P_{cr}$ ($\alpha_{cr} \geq 2$). EN 1993-1-1 clause 5.2.2 limits the method for frame applications where $\alpha_{cr} \geq 3$.

The global $P-\Delta$ effects, according to clause 5.2.1 of EN 1993-1-1 need to be considered for the cases where the value of $\alpha_{cr} \leq 10$ for an elastic global analysis, and $\alpha_{cr} \leq 15$ for a plastic global analysis. Global imperfections for frames are defined according to EN 1993-1-1 section 5.3.2. Basically, an initial frame rotation $\phi = h/200$ (where h is the height of the frame/structure) is recommended (Figure 1), although the value can be reduced based on the number of columns and height of the frame. If the applied horizontal loads in the frame are more than 15% of the vertical loads, clause 5.3.2 of EN 1993-1-1 allows the global imperfections to be neglected. In this circumstance, the effects of global imperfections are small compared to that of the applied horizontal loads.

To assess global instability in a structure, the problem is often addressed using the Finite Element Method. In simple terms, the stiffness of a beam element is reduced based on the level of axial force. The method leads to a stiffness matrix $[K]$ for the total structure, where the critical factor α_{cr} is obtained by solving the determinant $|K| = 0$. Different buckling modes can be found (eigenvalues). For global stability, local modes (related to individual members) are ignored. The exact answer for the problem is complex, leading to the implementation of simplified approaches. The exact answer for a simple beam with no axial or shear deformation is presented in Figure 7. The terms in the matrix depend on the stability functions ϕ_i . By necessity, simplification generally involves making approximation to the highly non-linear ϕ_i functions (see Figure 8), which in turn leads to recommendations regarding modelling.

At large values of N/P_{cr} , the difference between precise and approximate values for ϕ_i is significant. It is therefore recommended that individual members are modelled by at least 3 finite elements, which reduces the N/P_{cr} ratio by a factor of 9, and consequently reduces the error in taking approximated values for ϕ_i . The maximum value of N/P_{cr} is 4 (when $l_{eff} = 0.5l$), so modelling the member with 3 finite elements reduces the ratio to 0.44. As can be seen from Figure 8, the error between the approximate and precise values of ϕ_i functions for $N/P_{cr} = 0.44$ is insignificant.

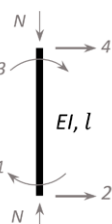
$$[K_{ij}^c] = \frac{EI}{l} \begin{bmatrix} 4\phi_3 & \frac{6\phi_2}{l} & 2\phi_4 & \frac{-6\phi_2}{l} \\ \frac{6\phi_2}{l} & \frac{12\phi_1}{l^2} & \frac{6\phi_2}{l} & \frac{-12\phi_1}{l^2} \\ 2\phi_4 & \frac{6\phi_2}{l} & 4\phi_3 & \frac{-6\phi_2}{l} \\ \frac{-6\phi_2}{l} & \frac{-12\phi_1}{l^2} & \frac{-6\phi_2}{l} & \frac{12\phi_1}{l^2} \end{bmatrix} \begin{matrix} \phi_1 = \beta \phi_2 \cotg(\beta) \\ \phi_2 = \frac{\beta^2}{3[1 - \beta \cotg(\beta)]} \\ \phi_3 = \frac{3}{4}\phi_2 + \frac{1}{4}\beta \cotg(\beta) \\ \phi_4 = \frac{3}{2}\phi_2 + \frac{1}{2}\beta \cotg(\beta) \\ \beta = \frac{\pi}{2} \sqrt{\frac{N}{P_{cr}}} \end{matrix}$$


Figure 7 – Formulation for the exact stiffness matrix^{8,9}.

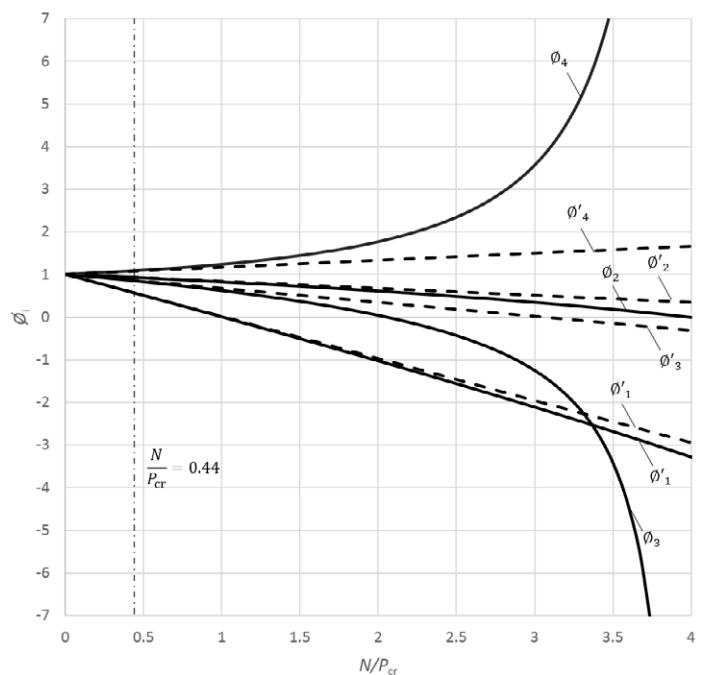


Figure 8 – Stability functions for exact (ϕ_i , solid lines) and for approximate (ϕ_i' , dashed lines) stiffness matrix stiffness (ϕ_i')^{8,9}.

Conclusions

- 1 Buckling problems demand the consideration of the deformed shape of the system;
- 2 The concept of an effective length is used to adapt the Euler buckling load to different boundary conditions;
- 3 An imperfect strut buckles before the plastic section capacity is reached;
- 4 Elastic section modulus must be used to back-calculate the initial imperfection;
- 5 Second order effects can be allowed for by using an amplification factor;
- 6 Approximate methods for stability functions ϕ_i are generally used in assessing frame stability;
- 7 Modelling with at least three finite elements per member reduces the error in using approximate stability functions.

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Stability and second order effects on steel structures: Part 2: design according to Eurocode 3

Ricardo Pimentel of the SCI illustrates the different methods provided by EN 1993-1-1 to address the topics of member stability, global frame stability and second order effects. Fundamental structural mechanics relating to stability was covered in Part 1.

Section 5.2 of EN 1993-1-1¹ introduces an approximate method to calculate the critical factor of frames (α_{cr}), based on the well-known Horne method² (Figure 1). The method is limited to frames with low axial force in the beams/rafters ($N_{Ed} \leq 0.10 N_{cr,R}$; N_{Ed} is the design axial load; $N_{cr,R}$ is the elastic critical load for buckling about the major axis of the beam/rafter) and for frames not steeper than 26°. For other cases, further guidance can be found in reference 3.

In section 5.2.2 of EN 1993-1-1, different methods are proposed to consider local ($P-\delta$) and global ($P-\Delta$) second order effects for structural analysis and member verifications. The following three main methods can be identified:

Method 1:

Both $P-\delta$ and $P-\Delta$ effects in addition to local and global imperfections are directly considered in the global analysis; the deformed structural shape is considered in the analysis, due to local and global imperfections and local and global second order effects; second order design internal forces are calculated. This design method may need to include in-plane and out of plane flexural buckling in addition to lateral torsional buckling.

Method 2:

$P-\Delta$ second order effects and global imperfections are considered in the structural analysis; $P-\delta$ effects are allowed for while performing stability checks according to EN 1993-1-1 section 6.3; the deformed structural shape is considered; second order design internal forces are calculated.

Method 3:

Both $P-\delta$ and $P-\Delta$ effects are accounted for when performing stability checks according to section 6.3 of EN 1993-1-1. In this method, an equivalent member length (effective length) needs to be defined. The allowance for $P-\Delta$ effects is made by increasing the $P-\delta$ effects by means of a longer member length. First order internal forces are considered for the member verification, which may include global imperfections – see EN 1993-1-1 5.3.2 (4). Global imperfections need to be included in the analysis, generally by applying the Equivalent Horizontal Forces (EHF).

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}} \right) \left(\frac{h}{\delta_{H,Ed}} \right)$$

H_{Ed} – Total storey shear;

V_{Ed} – Total vertical load at that storey;

h – Storey height;

$\delta_{H,Ed}$ – Horizontal displacement of the top storey relative to the bottom storey due to horizontal loads;

Buckling lengths greater than $2l$ may be required to allow for $P-\Delta$ effects in structures sensitive to those effects.

For **Method 1**, different approaches may be taken, as out-of-plane flexural buckling (FB) and lateral torsional buckling (LTB) may or may not be relevant. To allow for LTB, according to EN 1993-1-1 section 5.3.4, an equivalent bow imperfection equal to $k \cdot e_{0,d}$ may be used, where $e_{0,d}$ is the equivalent bow imperfection of the weak axis of the profile and k is a correction factor; it is also stated that in general, torsion imperfections need not to be considered. According to the UK National Annex⁴, the value of k is to be taken as 1. The application of Method 1 is more often used in research, but several commercial software packages already allow users to directly consider the $P-\delta$ and $P-\Delta$ effects within the structural analysis. Method 1, where local and global imperfection are directly considered in the analysis, is necessary for the cases where the following condition are met (clause 5.3.2 (6) of EN 1993-1-1):

- $\alpha_{cr} < 10$, for elastic global analysis;
- At least one moment resisting joint at one member end;
- $N_{Ed} > 0.25 N_{cr,0}$, where N_{Ed} is the design axial load and $N_{cr,0}$ the critical load assuming a pin-ended strut. This means that for a simple column system,

$$\alpha_{cr} = \frac{N_{cr,0}}{N_{Ed}} < 4.$$

Method 2 can be implemented by two possible approaches:

- **Method 2.1** – Considering the $P-\Delta$ effects directly through a numerical geometric non-linear global analysis considering global imperfections; usually computed by commercial software packages; this may increase the required analysis time for large frames and multiple load combinations;
- **Method 2.2** – Considering the $P-\Delta$ effects indirectly by amplifying the first order sway effects (including global imperfections) by the so-called amplification factor $k_{sw} = \frac{1}{1-1/\alpha_{cr}}$. As introduced in Part 1⁵, this method is

limited to the cases where $\alpha_{cr} \geq 3$. For multi-storey buildings, the rule may be used when vertical and horizontal loads and frame stiffness are similar between storeys – see EN 1993-1-1 5.2.2 (6) B.

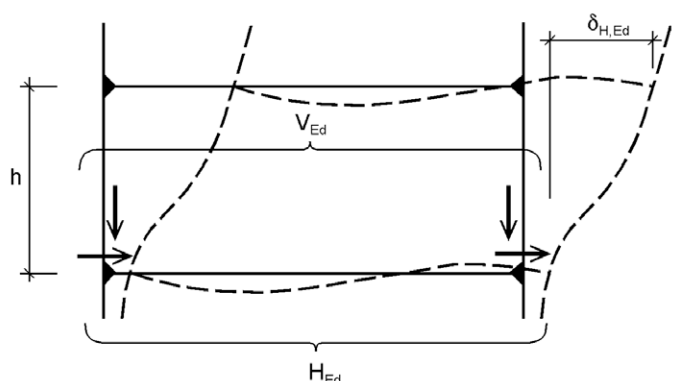


Figure 1 – Horne method to calculate α_{cr} of frames.

Both methods 2.1 and 2.2 are extensively used in practice. When verifying members according to EN 1993-1-1 section 6.3, system length should be used as the buckling length.

In **Method 3**, the designer must determine an appropriate effective length that allows for the consideration of $P-\Delta$ effects while performing member checks according to section 6.3 of EN 1993-1-1. As the design is based on first order internal forces, the complexity of the analysis is removed, but the effective length needs to be specified for each column. The concept of effective length was introduced in Part 1 of the current article for isolated struts, where the horizontal or rotational restraints of the strut ends were assumed as infinitely rigid. This does not represent reality: (i) rotational stiffness of the nodes is related to the flexural stiffness of the elements that are connected to the nodes, resulting in a rotational spring on each node – $k_{r,i}$ (Figure 2); (ii) if a structure is susceptible to second order global effects, the complexity is increased, as the structure is horizontally flexible (assessed by the value of α_{cr}), resulting in horizontal springs on each node – $k_{h,i}$ (Figure 2).

When a column is integrated in a frame, the concept of effective length may be described as the fictional pin-ended strut length that buckles at the same time as the frame for a specified load case⁶. Based on the value of α_{cr} for the entire frame, the critical load N_{cr} for each column can be calculated by multiplying the design axial load on each column by the value of α_{cr} . The effective lengths can then be obtained by a back calculation, knowing that $N_{cr} = (\pi^2 EI) / (l_{eff})^2$. Thus, the effective length of a column is dependent on the applied load and spring stiffness at the nodes. The values of l_{eff} obtained are only appropriate within the load arrangement assumed to calculate α_{cr} . This method is described in Annex E.6 of BS 5950-1⁷.

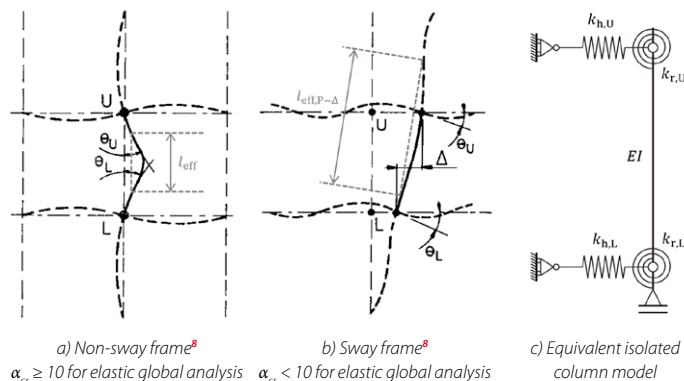


Figure 2: Effective length concept in sway and non-sway frames

In practice, while using **Method 3**, the definition of the effective buckling length is often obtained indirectly by a simplified analysis where each column is considered individually, with no dependency on the applied load. There are several resources to assess the problem, such as the well-known Wood method⁹, which provides effective buckling lengths for sway or non-sway frames. These approximate methods are intended to provide an

answer for the problem shown Figure 2c. The Wood method can be found in Annex E of BS 5950-1 as well as in NCCI SN008a¹⁰. Based on the model in Figure 2c, simplified methods usually assume that $k_{h,L} = \infty$ and $k_{h,U} = 0$ ⁹.

The approximate methods provide exact results if every member has the same rigidity parameter $\Phi_r = \sqrt{EI/N_{Ed}}$ where EI is the flexural stiffness of the column, N_{Ed} is the design axial load on the column, and l is the system length of the column⁶. This means that all columns would buckle at the same time. The columns with low values of Φ_r are the critical members (members which induce frame instability), for which the method gives conservative values of the buckling length. For members with high values of Φ_r , buckling lengths are unconservative. For the critical members, the method can be seen as a conservative approximation for the critical load of the frame⁶.

The approximate methods provide an efficient and systematic procedure to assess the problem. However, the following effects/simplifications are usually disregarded/considered in the process^{6,8,9,11,12}: (i) only columns are affected by $P-\Delta$ effects, while internal forces to design other elements (beams, connections) will be always based on first order theory; (ii) for frames sensitive to second order effects, the effective lengths calculated are the same for any value of α_{cr} ; (iii) there is no influence of the applied load; (iv) for columns in non-sway frames, the rotation at opposite ends of the restraining elements are equal in magnitude and opposite in direction, producing single curvature bending; (v) for columns in sway frames, the rotation at opposite ends of the restraining elements are equal in magnitude and opposite in direction, producing double curvature bending; (vi) all columns buckle simultaneously; (vii) stiffness parameter Φ_r is the same for all columns; (viii) no significant axial force exists in the beams; (ix) all joints are rigid; (x) joint restraint is distributed to the column above and below the joint in proportion to EI/l for the two columns. Further information about approximate methods can be found in reference 11.

Two worked examples follow, where the results obtained from the application of methods 2.1, 2.2 and 3 are compared.

Worked example 1: simple column

Influence of the number of finite elements on simple struts (Table 1):

The results support the conclusions from Part 1: for low values of N_{Ed}/N_{cr} the errors in using an approximate stiffness matrix are less significant than for cases where N_{Ed}/N_{cr} is close to 4. The consideration of 3 finite elements for the strut gives reasonable results for the four cases.

The design of the column based on **Method 2** (2.1 by a numerical $P-\Delta$ or 2.2 considering k_{sw}) and **Method 3** will be undertaken for the structure in Figure 3. Two examples are considered for different levels of horizontal load. A comparison of the Unity factor (UF) for relevant checks according to EN 1993-1-1 is presented in Table 2.

From Worked Example 1, it can be noted that there is very close agreement in the utilization factor between methods 2.1 and 2.2. Method 3 is conservative for $N_{Ed} = 75$ kN and $H/2 = 10$ kN. If the horizontal load $H/2$ is increased to 20 kN, Method 3 becomes unconservative.

Boundary conditions	Section	l [m]	Theoretical value of $N_{cr,z}$ [kN]	$N_{cr,1}$ 1 FE	$N_{cr,3}$ 3 FE	$N_{cr,5}$ 5 FE	$N_{cr,10}$ 10 FE
Cantilever	254 UC 107	10	$N_{cr,z} = \frac{\pi^2 EI}{(2l)^2} = 307.16$	309.47	307.19	307.17	307.16
Pinned Pinned	254 UC 107	10	$N_{cr,z} = \frac{\pi^2 EI}{l^2} = 1228.65$	1493.86	1230.59	1228.91	1228.66
Pinned Fixed	254 UC 107	10	$N_{cr,z} = \frac{\pi^2 EI}{(0.6992l)^2} = 2513.18$	3734.64	2528.93	2515.68	2513.64
Fixed Fixed	254 UC 107	10	$N_{cr,z} = \frac{\pi^2 EI}{(0.5l)^2} = 4914.59$	∞^*	5022.36	4930.35	4915.65

* - See Part 1⁵, Figure 8; this example represents $N_{Ed}/N_{cr} = 4$;

Table 1: Buckling analysis of a strut considering different number of finite elements (FE)¹³.

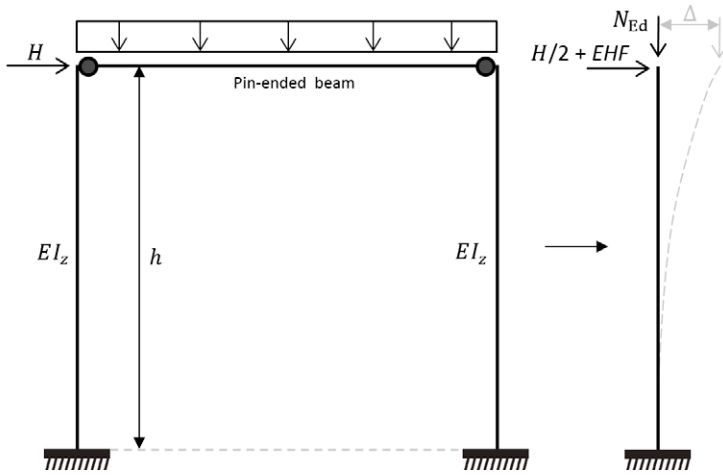


Figure 3: Example of a slender simple column.

$h = 10 \text{ m};$
 Columns: 254 UC 107; $I_z = 5928 \text{ cm}^4$; Steel: S355 JR
 $N_{Ed} = 75 \text{ kN}$; Example 1.1: $H = 20 \text{ kN}$;
 Example 1.2: $H = 40 \text{ kN}$ (factored loads)
 EN 1993-1-1 section 5.3.2:
 $EHF = \frac{1}{200} * 75 = 0.375 \text{ kN}$
 $N_{cr} = \frac{\pi^2 EI_z}{(2l)^2} = 307.16 \text{ kN}$
 If the system is represented by a single column:
 $\alpha_{cr} = \frac{N_{cr}}{N_{Ed}} = \frac{307.16}{75} = 4.10$
 As $\alpha_{cr} > 4$, local bow imperfections can be disregarded in the analysis – EN 1993-1-1 5.3.2 (6).
 $k_{sw} = \frac{1}{1-1/\alpha_{cr}} = \frac{1}{1-1/4.10} = 1.32$

Worked example	Method	l_{eff} [m]	N_{Ed} [kN]	$H/2 + EHF$ [kN]	First order bending moment [kNm]	Second order bending moment [kNm]	UF
1.1	2.1	10	75	10.375	103.75	131.29	0.53
	2.2	10	75	10.375	103.75	$103.75 * 1.32 = 136.95$	0.55
	3	20	75	10.375	103.75	-	0.63
1.2	2.1	10	75	20.375	203.75	257.77	1.04
	2.2	10	75	20.375	203.75	$203.75 * 1.32 = 268.95$	1.09
	3	20	75	20.375	203.75	-	0.96

Table 2: Results for two different load arrangements: simple column¹³.

Worked example 2: three-storey frame

In worked example 2, the comparisons are extended to a three-storey frame (shown in Figure 4). Geometric conditions can be found in Table 3. Two examples are considered for different levels of horizontal load. Comparisons of the Unity factor (UF) for relevant checks according to EN 1993-1-1 are presented in Table 4 and Table 5 for the two horizontal load arrangements.

The effective buckling lengths were obtained by a back-calculation based on the global buckling mode of the frame. Example for Model 4:

$$N_{cr,AB} = \alpha_{cr} N_{Ed,AB} = \pi^2 EI_{z,AB} / (l_{eff,AB})^2 \text{ so, } l_{eff,AB} = \sqrt{\frac{\pi^2 EI_{z,AB}}{5.87 * 4055.47}} = 4.23 \text{ m}$$

Note: this process was adopted to obtain as much precision as possible in the comparison between the methods. It should be highlighted that the back-calculation method based on α_{cr} is only valid for the considered

load arrangement. Conservative results for the effective lengths are expected when using approximated methods which are valid for any load arrangement.

The numerical consideration of global $P-\Delta$ effects and the approximate consideration of those effects with the amplification factor show a very close agreement in the utilization factor (as for worked example 1). The effective length method still gives a reasonable answer in comparison to the other two methods, but differences around 0.15 in the utilization factor (conservative or non-conservative) can be obtained.

Influence of the number of finite elements on frame stability:

The differences in modelling precision are demonstrated in Figure 5, which shows the different buckling modes and values of α_{cr} for models with 1 and 10 finite elements per member (using Model 4 from worked example 2.1). The non-sway frame has horizontal supports on each floor level.

Model	Bases	Beams	Columns	I_z [mm ⁴]	S [m]	h_1 [m]	h_2 [m]	h_3 [m]	$0.25N_{cr,0,AB}$ [kN]
1	Pinned	UB 457 191 161	UC 356 406 551	82670	10	3.75	3.00	3.00	30461.02
2	Pinned	UB 457 191 161	UC 356 406 340	46850	10	4.00	3.20	3.20	15172.20
3	Fixed	UB 457 191 161	UC 356 406 235	30990	10	5.00	4.00	4.00	6423.04
4	Fixed	UB 457 191 161	UC 356 368 177	20530	10	5.00	4.00	4.00	4255.08

Vertical loads on each storey (unfactored): self-weight; permanent loads: 50 kN/m; imposed loads: 35 kN/m;
 Horizontal loads: Example 2.1: $H = EHF$; Example 2.2: $H = EHF + 100 \text{ kN}$ (imposed load, unfactored) on each storey;
 $EHF: \theta = 1/200$; Column spacing: 10 m; $h_1/h_2 = h_1/h_3 = 1.25$; Material: S355 JR;
 Columns under minor axis bending; Beams under major axis bending; 10 Finite elements per member;
 The solution for Model 4 was configured to achieve $N_{Ed} > 0.25 N_{cr,0}$ (clause 5.3.2 (6) of EN 1993-1-1).

Table 3: Models considered in worked example 2.

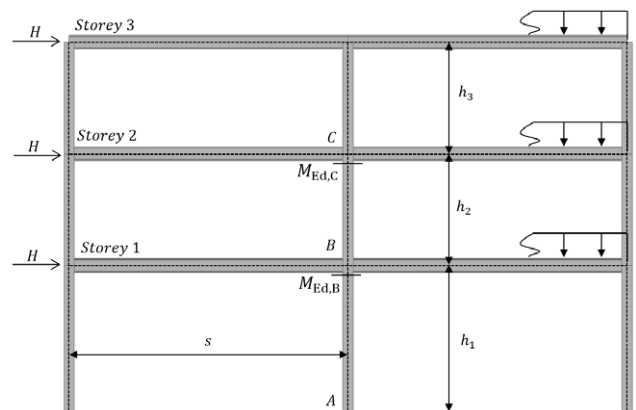


Figure 4: Geometry for worked example 2.

Note: in real design cases, perfectly fixed bases are not realistic. Nominally fixed bases may be assumed with the flexural stiffness of the base equal to the flexural stiffness of the column⁷.

Model	α_{cr}	k_{sw}	Design method	$M_{Ed,B}$ [kNm]	$M_{Ed,C}$ [kNm]	$N_{Ed,AB}$ [kN]	$N_{Ed,BC}$ [kN]	$l_{eff,AB}$ [m]	$l_{eff,BC}$ [m]	UF_{AB}	UF_{BC}
1	7.33	1.16	2.1	60.84	33.60	3860.91	2549.80	3.75	3.00	0.21	0.13
			2.2	61.04	34.85	3860.53	2549.54	3.75	3.00	0.21	0.13
			3	52.62	30.04	3860.53	2549.54	7.78	9.58	0.30	0.24
2	4.42	1.29	2.1	70.66	33.19	3906.91	2585.93	4.00	3.20	0.36	0.21
			2.2	72.01	36.34	3906.07	2585.50	4.00	3.20	0.36	0.22
			3	55.83	28.17	3906.07	2585.51	7.50	9.22	0.49	0.39
3	8.25	1.14	2.1	35.71	26.90	3989.55	2645.51	5.00	4.00	0.52	0.32
			2.2	35.93	27.84	3988.02	2644.78	5.00	4.00	0.52	0.32
			3	31.52	24.42	3988.02	2644.78	4.42	5.43	0.49	0.36
4	5.87	1.21	2.1	38.17	26.49	4057.78	2693.40	5.00	4.00	0.64	0.40
			2.2	39.00	28.35	4055.47	2692.36	5.00	4.00	0.64	0.40
			3	32.23	23.43	4055.47	2692.37	4.23	5.19	0.66	0.49

Table 4: Worked example 2.1: horizontal loads with only EHF¹³.

Model	α_{cr}	k_{sw}	Design method	$M_{Ed,B}$ [kNm]	$M_{Ed,C}$ [kNm]	$N_{Ed,AB}$ [kN]	$N_{Ed,BC}$ [kN]	$l_{eff,AB}$ [m]	$l_{eff,BC}$ [m]	UF_{AB}	UF_{BC}
1	7.31	1.16	2.1	821.33	453.69	3861.26	2549.73	3.75	3.00	0.48	0.26
			2.2	823.99	470.48	3860.86	2549.48	3.75	3.00	0.48	0.27
			3	710.34	405.58	3860.81	2549.49	7.79	9.59	0.55	0.36
2	4.41	1.29	2.1	953.98	448.26	3907.42	2585.94	4.00	3.20	0.87	0.41
			2.2	972.20	490.65	3906.37	2585.45	4.00	3.20	0.88	0.43
			3	753.64	380.35	3906.29	2585.47	7.51	9.23	0.96	0.57
3	8.23	1.14	2.1	482.07	363.31	3990.08	2645.67	5.00	4.00	0.80	0.51
			2.2	485.11	375.79	3988.33	2644.74	5.00	4.00	0.81	0.52
			3	425.53	329.64	3988.29	2644.74	4.42	5.43	0.75	0.54
4	5.86	1.21	2.1	515.31	357.77	4058.47	2693.69	5.00	4.00	1.09	0.69
			2.2	526.53	383.70	4055.75	2692.33	5.00	4.00	1.14	0.71
			3	435.14	316.28	4055.70	2692.34	4.23	5.19	0.98	0.71

Table 5: Worked example 2.2: horizontal loads with EHF + 100 kN (imposed load, unfactored)¹³.

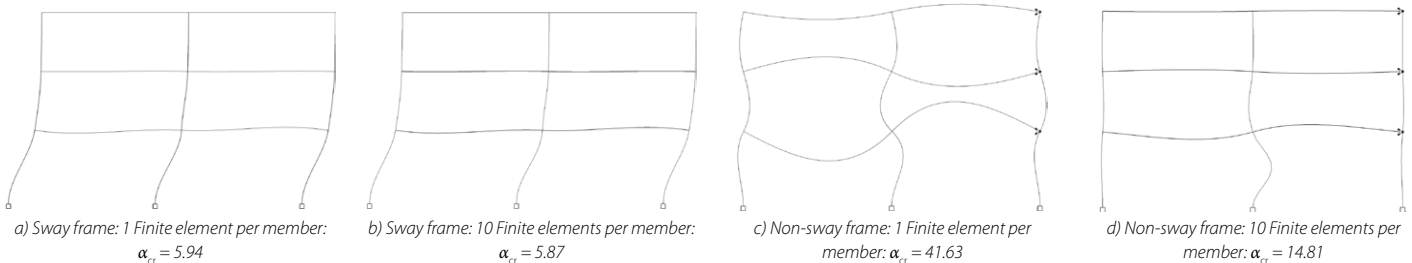


Figure 5: Influence of the number of finite elements per member on frame stability¹³.

Calculation of α_{cr} using the Horne method:

For model 4 of worked example 2.1, the calculation of α_{cr} according to clause Section 5.2 of EN 1993-1-1 is shown in Figure 6. The approximate

value of 6.61 may be compared with the precise value of 5.87 from Table 4 and 5.86 from Table 5. The approximated value of 6.61 is the same for worked examples 2.1 and 2.2, as the ratio $H_{Ed}/\delta_{H,Ed}$ is identical in the method.

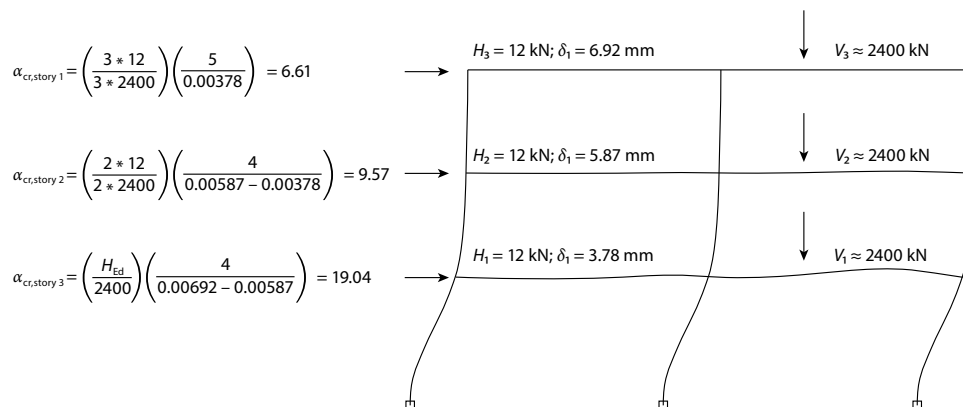


Figure 6: Calculation of α_{cr} with the Horne method (worked example 2.1)¹³.

Conclusions

- 1 Eurocode 3 provides essentially 3 different methods to consider local and global second order effects when verifying members;
- 2 In practice, local second order effects are usually considered when checking member stability according to section 6.3 of EN 1993-1-1;
- 3 Local imperfections may need to be considered for global analysis; this may be mandatory according to clause 5.3.2 (6) of EN 1993-1-1; the criteria is more significant for frames with fixed bases where lower α_{cr} can be obtained with slender members;
- 4 The effective length method considers the effects of global second order effects by increasing the local second order effects; buckling lengths greater than $2l$ may be required;
- 5 The numerical consideration of global $P-\Delta$ effects and the approximated consideration of those effects with the amplification factor give very similar results; For member stability verifications according to section 6.3 of EN 1993-1-1, system lengths should be used;
- 6 The effective length method gives a reasonable answer in comparison to the other two other methods where second order internal forces are calculated. Differences between methods can be up to approximately 0.15 in the utilization factor (conservative or non-conservative); differences are less significant for higher values of α_{cr} .
- 7 The importance of considering more than 1 finite element per member was demonstrated for struts and frames. At least 3 finite elements are recommended;
- 8 Horizontal loads have a small influence in the values of α_{cr} .

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The design of crane girders

Recent correspondence in *Verulam*¹ suggested that there were no decent examples of crane girder design to the Eurocodes. David Brown of the SCI rises to the challenge...

The problem

According to the contribution in *Verulam*, a number of problems exist with the design of a mono-symmetric member (a plate welded to the top flange of a UB) and destabilising loads:

- BS 5950 examples have 'mysteriously disappeared' from the equivalent Eurocode publications.
- The only way to design the member is to use 'a piece of software from a French website'.
- There is no way of checking the result (from the French software).
- Gantry girders would have to be doubly symmetric, or have the top flange fully restrained.

What are the options?

Looking back at the BS 5950 examples in the SCI library, most are mono-symmetric with a channel welded to the top flange. An example with a plain plate welded to the top flange is presented in early editions of the 'Red Book'².

Some of the examples calculate the section properties of the compound section – not a precise task, (especially before channels had parallel flanges) and verify the fabricated member on that basis. Alternative examples adopt the traditional and simpler approach of assuming that the additional plate (or channel) carries the horizontal loads, and the rolled section carries the vertical loads.

If one held the pessimistic expectation that the Eurocodes always adopt the most complex approach, one might be pleasantly surprised to find that the simple approach is allowed in clause 5.6.2(4) of EN 1993-6, which is the Standard covering the design of crane supporting structures. According to this clause, lateral loads are resisted by the top flange, and vertical loads are resisted by the main beam under the rail. This simple approach will be familiar, and facilitates the use of mono-symmetric sections.

Following this simple approach, torsional moments are resisted by a couple acting horizontally on the top and bottom flange. As an alternative, torsion may be treated rigorously.

Lateral-torsional buckling

Gantry girders are unrestrained, and have lateral loads applied at the top flange level (or above). As the beam buckles, the vertical loads may be eccentric to the shear centre, so there are additional torsions on the section, as indicated in Figure 1. Clause 6.3.2.1 of EN 1993-6 insists (quite properly) that these torsions must be accounted for. The designer

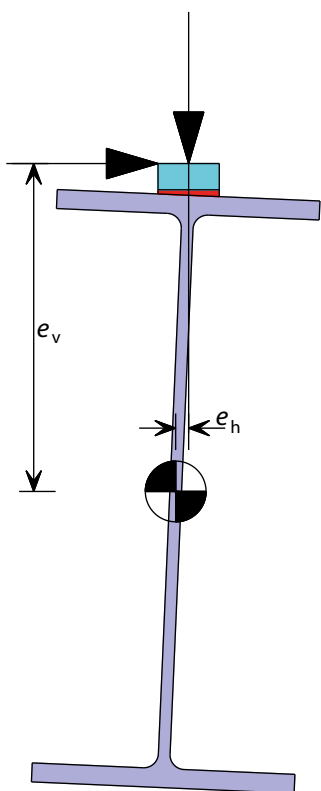


Figure 1 Torsions on a gantry girder

again has options, according to clause 6.3.2.3.

The first option is to simply consider the top flange and part of the web acting entirely alone, and check it as a simple strut. Safe, certainly, but conservative. The second option is to assess the member for the combined effects of lateral-torsional buckling, minor axis moment and torsion, using the interaction expression presented in Annex A of the Standard. The UK National Annex endorses the use of this alternative.

Of course, the interaction expression looks complicated:

$$\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk} / \gamma_{M1}} + \frac{C_{Mz} M_{z,Ed}}{M_{z,Rk} / \gamma_{M1}} + \frac{k_w k_{z,w} k_{\alpha} B_{Ed}}{B_{Rk} / \gamma_{M1}} \leq 1$$

A numerical worked example would help, as the correspondence in *Verulam* notes. Fortunately there is a full worked example in P385³, which is SCI's publication on the design of steel beams in torsion. Example 2 is precisely the case under consideration – a gantry girder, except the selected member is a UB with no plate. Because this comprehensive numerical example exists, no further attention is paid to the interaction expression in this article.

Destabilising loads

Loads that move with the buckling compression flange are classed as destabilising. As the correspondence in *Verulam* indicates, one would normally assume that gantry girders are subject to destabilising loads.

EN 1993-6 offers an interesting twist (no pun intended) to the classification of destabilising loads. Clause 6.3.2.2 suggests that if the crane rail is fixed directly to the runway beam, the applied vertical load can be considered as stabilising. This unexpected conclusion is because, as shown in Figure 2, as the runway beam starts to twist, the application of load moves to the 'high' side of the rail, which is actually on the 'restoring' side of the shear centre. Thus the load is stabilising and in these circumstances the Standard notes that it may be assumed that the loads are applied at the shear centre.

If the rail is supported on a flexible elastomeric pad, the loads are destabilising and the Standard notes that the loads should be assumed to be applied at the top of the flange.

In BS 5950, destabilising loads were treated by multiplying the system

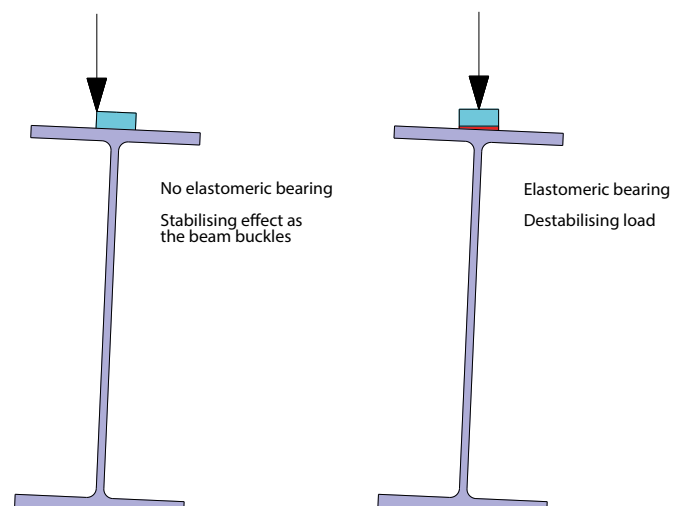


Figure 2 Influence of crane rail on load classification

length by 1.2 (typically), with further adjustment depending on the support conditions. The equivalent uniform moment factor m_{LT} had to be taken as 1.0 (so no benefit from the shape of the bending moment diagram). The Eurocode deals with destabilising loads by adjusting the calculated value of M_{cr} , which will lead us to the comment about using software from a French website.

Calculation of M_{cr}

The background to the problem of M_{cr} is that BS 5950 presents bending strengths p_b for different values of slenderness, λ_{LT} , which is very convenient for the designer, as long as one is not interested how the values have been derived. If interest is sparked, Annex B of BS 5950 provides the background. With patience and algebraic dexterity, one can demonstrate that the BS 5950 terms depend on a familiar friend – the elastic critical buckling moment, M_{cr} . This has been discussed previously⁴.

M_{cr} can be calculated using a formula. The version of the formula which allows for destabilising loads is perfectly amenable to computation by paper, pencil and calculator as the *Verulam* correspondence wished. Software solutions merely make the process easier and, many would say, less open to error. After extensive experience asking course delegates to complete a manual calculation of M_{cr} even without destabilising loads, the conclusion is that generally over 80% fail to compute the correct answer. Sadly, the main problem is that delegates attempt to use inconsistent units within the calculation. Maybe software is safer after all.

The French software mentioned is *LTBeam*, which has been discussed several times. Despite the assertion in *Verulam*, independently written software from the UK (does that make it better?) exists and is freely available at steelconstruction.info

If necessary, these two programs could be used for mutual checking, and then proved by hand calculation – though a spreadsheet is strongly recommended to remove the tedium of the latter option.

How to check?

The calculation of M_{cr} is merely a step on the way to the result, so checking of the final resistance is probably wise. Options are available, starting with a 'sense check' against the results from BS 5950. Since the introduction of the Eurocodes the consistent message has been that the structural mechanics has not changed, so one would not expect to find significant differences in the results obtained by either code. Generally, the LTB resistance according to the Eurocode is a little higher than according to BS 5950, so that needs to be recognised, as well as taking $m_{LT} = 1.0$ for destabilising loads.

The wise authors of BS 5950 recognised that increasing the effective length of the member was a good way to allow for destabilising loads. That simple check can be completed by looking at the calculated member resistances for the two lengths.

Simple design assessment

Some straightforward checks of the example presented in P385 have been completed. The example demonstrates the verification of a member subject to combined major and minor axis bending combined with torsion, but if the example is reconfigured to assume lateral loads (and torsional effects due to eccentricity) are taken by a plate welded to the top flange, the exercise becomes a review of the main member.

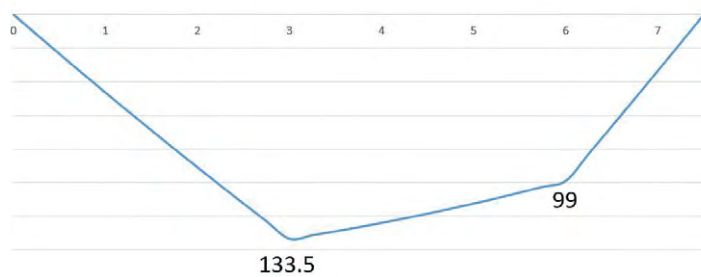


Figure 3 Bending moment diagram

The vertical loads are destabilising, so according to EN 1993-6 are assumed to be applied at the level of the top flange. Accounting for the position of the loads, $M_{cr} = 320 \text{ kNm}^*$, according to P385, and $M_b = 277 \text{ kNm}^*$.

The span of the gantry girder is 7.5 m, so applying a factor of 1.2 results in a span of 9 m. Then one must make a reasonable estimate of the shape of the bending moment diagram, or conservatively assume that $C_1 = 1.0$

Looking at the bending moment diagram (Figure 3), it looks vaguely similar to that for a UDL, admittedly with some angularity, but for a quick check, assume that $C_1 = 1.13$, mainly for easy use of the look-up tables in the Blue Book.

For the trial section of a $533 \times 210 \times 101 \text{ UB}$ in S275 (note that all beams are S355 nowadays!), a buckling length of 9 m and $C_1 = 1.13$, the buckling resistance $M_b = 288 \text{ kNm}$. As a coarse check, this is quite reassuring when compared to the computed value of 277 kNm^* .

A further approach is to use the look-up tables in the back of P362⁵, where χ_{LT} depends only on h/t_f and L/i_z , which more mature designers will recognise as D/t and L/r_{yy} in previous nomenclature. The tables in P362 assume $C_1 = 1.0$, so are likely to deliver a smaller resistance than computed with precision.

$$h/t_f = 536.7/17.4 = 31$$

$$L/i_z = 9000/45.7 = 196$$

Using Table E2 from P362, $\chi_{LT} = 0.38$ with some approximate interpolation.

$$\text{Therefore } M_b = 0.38 \times 2610 \times 10^3 \times 265 \times 10^{-6} = 262 \text{ kNm}$$

This seems to offer reassurance that we are in the correct parish, at least, when compared to the computed value of 277 kNm^* .

What has not been addressed!

In the opinion of the author, the challenge with gantry girders is not in fact the member verification, but the determination of the applied actions in accordance with EN 1991-3, a problem which was not mentioned in *Verulam*. A treatise on the subject is available for download⁶, but the topic is complex.

Other issues not addressed here are the deflection limits for crane supporting structures, which may be more important than the member resistance. Designing the supporting structure to control the spread of the gantry beams will be important. Finally, fatigue design may govern the size of the member – an introduction to the subject⁷ and example calculations⁸ have been published in NSC.

*Footnote

Readers trying to replicate the calculation of M_{cr} as quoted in P385 may have some difficulty. The correct value of M_{cr} appears to be between 336 and 340 kNm and consequently $M_b = 288 \text{ kNm}$. Although it would be tempting to blame the software, it appears the user calculated the level of load application as $533/2 + 65 = 331 \text{ mm}$, when 286 mm should have been used (the load is applied at the top flange, not on top of the 65 mm rail).

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Fatigue of bracing in buildings

BS5950 states that buildings subject to fluctuating wind loads do not need to be checked for fatigue but EC3 contains no such statement. Richard Henderson of the SCI considers the issues and illustrates a fatigue check of wind bracing in a conventional building.

Introduction

Clause 2.4.3 Fatigue in BS5950-1:2000, a code specifically for the design of steelwork in buildings, states "Fatigue need not be considered unless a structure or element is subjected to numerous significant fluctuations of stress. Stress changes due to normal fluctuations in wind load need not be considered". The ANSI/AISC 360-16 Specification for structural steel buildings Chapter B clause 11 states "... Fatigue need not be considered ... for the effects of wind loading on typical lateral force-resisting systems ...". BS EN 1993-1-1 and BS EN 1993-1-9 (Part 1-9) include no such clause but BS EN 1993-1-1 forms the foundation for a series of codes for the design of bridges, towers and other structures. Bridges are routinely checked for fatigue. Other structures such as chimneys and masts may be subject to wind-induced oscillations and need to be checked for fatigue.

The connections at the ends of wind bracing are often made using gusset plates, fillet welded to end plates and beam flanges. Tubular tension/compression bracing members may have bolted spade-end connections fillet welded to end plates.

Fatigue Strength Curves

An introduction to fatigue design was published in NSC magazine last year. Part 1-9 clause 7.1 gives the fatigue strength for nominal stress ranges for a range of details, identified in Tables 8.1 to 8.10. The fatigue strength is defined by a $(\log \Delta \sigma_r) - (\log N)$ curve for each detail category as shown in Figure 1. For a constant amplitude nominal stress range, the curve gives the number of cycles to failure or endurance. The curve number is the detail category and is the constant amplitude nominal stress range that will result in failure after 2 million cycles. The curves change in slope at $N = 5$ million cycles. For nominal stress ranges lower than a certain value known as the cut-off limit $\Delta \sigma_L$, fatigue damage is considered not to occur. The curves are based on the results of tests on large-scale specimens collected over several decades.

Fatigue damage can be calculated for a given detail using the relevant

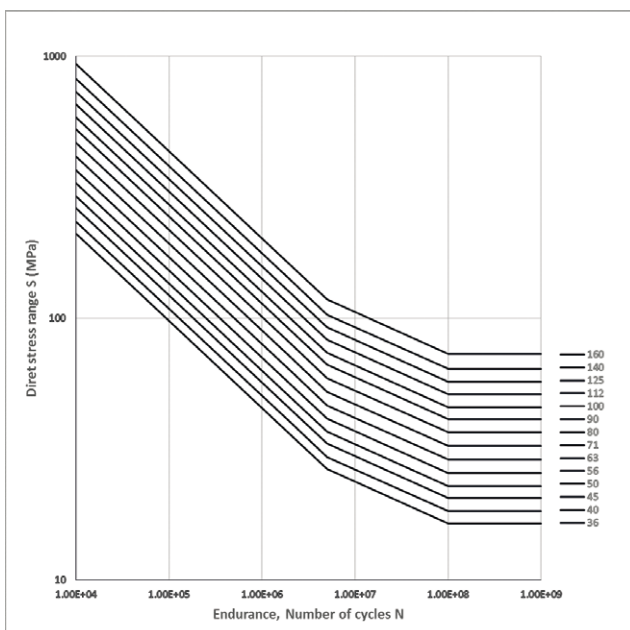


Figure 1: Fatigue strength curves

fatigue curve from Part 1-9 to determine the number of cycles to failure N_i for a given stress range i and using Miner's summation for fatigue damage $\sum n_i/N_i$ where n_i is the number of occurrences of this stress range over the life of the structure. The fatigue damage should be less than or equal to 1.0 for the detail to be acceptable (see Part 1-9, Annex A clause A5). Some fillet welded details are in the lowest classes of detail identified in Part 1-9 Table 8.5: either detail category 36* or 40.

Wind loads

BS EN 1991-1-4 Annex B includes a graph of the number of times in 50 years that a wind gust load equals or exceeds a given proportion of the once in 50 year gust load, expressed as a percentage (see Figure 2). This curve is introduced in Annex B for use in the procedure for determining the structural factor $c_s c_d$ in wind load calculations. BS EN 1991-1-4 gives no guidance on the use of the curve for fatigue calculations due to gust loads.

The relationship between the quantities is given as:

$$\frac{\Delta S}{S_k} = 0.7 (\log_{10}(N_g))^2 + 17.4 \log_{10}(N_g) + 100$$

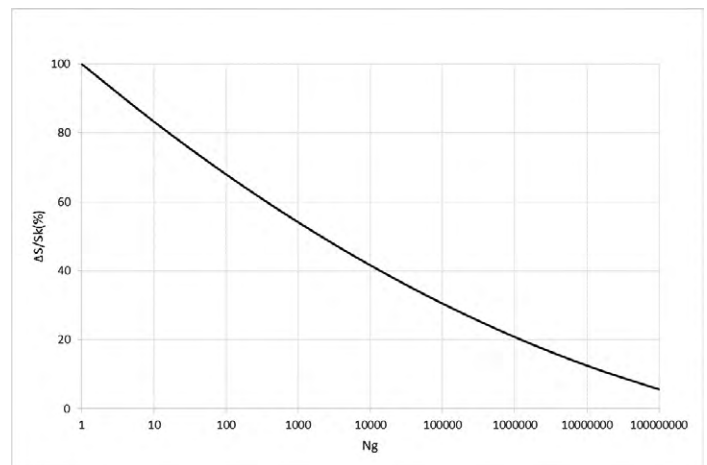


Figure 2: Number of gust loads N_g during a 50 year period

This graph provides the spectrum of stress ranges to which a detail is subjected. An unconsidered examination of the graph suggests that a load equal to 15% of the once in 50 year load ($\Delta S/S_k = 15\%$) occurs about 5 million times during the 50-year design life of the building. If the design load results in a stress equal to yield, a serviceability stress range of 36 MPa (about $355 \times (0.15/1.5)$) occurs enough times to cause a fatigue failure in a class 36 joint, for which a constant amplitude stress range of 36 MPa causes failure after 2 million cycles.

A crude examination such as this neglects a proper assessment of the stress ranges to which the bracing connection details are subjected. The bracing members are usually designed for wind loads and equivalent horizontal forces (EHF). These forces may also be amplified by a factor based on the elastic critical load factor of the building. Fatigue is a serviceability load case and the load factor on the wind load is therefore equal to unity instead of 1.5. Also, the EHF and amplification factor are intended to allow for global imperfections and second order effects respectively and are therefore not included in fatigue calculations. The stress ranges for the fatigue check are therefore significantly smaller than might initially be imagined.

Design Example

An example fatigue check on a connection detail for a bracing member taken from the design example in SCI's publication P365 Steel building design: medium rise braced frames is illustrative.

The ultimate design load in the bracing member from ground to first floor is 539 kN. 60.9% of this force is due to wind load and it includes an amplification factor of 1.17. The serviceability load due to wind alone is therefore:

$$\frac{537.4}{1.17} \times 0.609 \times \frac{1}{1.5} = 187.2 \text{ kN}$$

The bracing member chosen is a 168 x 6.3 CHS in S355 material. A Tee or spade end connection is adopted and the double-sided fillet weld between the end plate on the tube and the projecting plate is designed in accordance with clause 7.6 of BS EN 1993-1-8 which determines the effective lengths of the weld. If the welds are sized according to the design load, as allowed in clause 7.3.1(6) of BS EN 1993-1-8, 8 mm leg fillet welds are adequate (weld throat = 5.7 mm). The connection detail is illustrated in Figure 3.

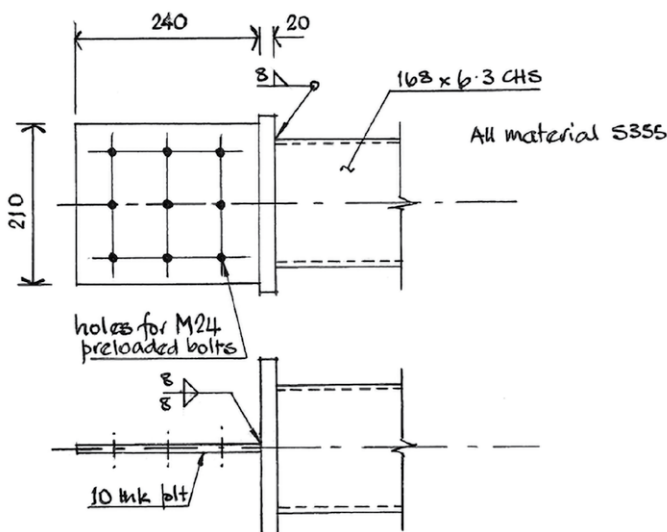


Figure 3: Bracing connection

Fatigue Check

Checks on two welds are necessary for the end connection: the tube to end plate and the end plate to spade end welds. The relevant detail categories are 40 and 36*; the latter category has a modified curve in accordance with clause 7.1(3) Note 3 in Part 1-9. The curves are shown in Figure 4.

Fatigue damage is defined in Annex A para. A.5 of Part 1-9 as:

$$D_d = \sum_i^n \frac{n_{Ei}}{N_{Ri}}$$

where n_{Ei} is the number of cycles associated with the stress range $\gamma_{FF} \Delta \sigma_i$ for band i in the factored spectrum and N_{Ri} is the endurance in cycles from the fatigue strength curve for a stress range of $\gamma_{MF} \gamma_{FF} \Delta \sigma_i$. According to the UK National Annex, $\gamma_{MF} = 1.1$ and $\gamma_{FF} = 1.0$.

The factored stress range spectrum is found from Figure 2. Stress ranges

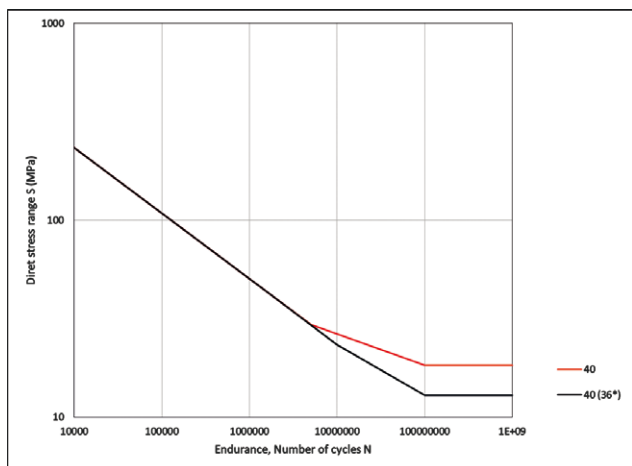


Figure 4: Fatigue strength curves

$\Delta \sigma_i$ corresponding to equal intervals of $\log_{10} N_g$ along the horizontal axis are considered in calculating the fatigue damage. The values of N_g range between 1.0 at 100% of S_k multiplied by the partial factors and the value of N_g at the factored cut-off limit $\Delta \sigma_L$. 100 intervals are chosen to achieve good convergence. The number of cycles n_{Ei} of the occurrence each stress range is calculated from the spectrum and the number of cycles to failure N_{Ri} for the stress range is calculated from the fatigue strength curve (Figure 3). The ratio of n_{Ei}/N_{Ri} is summed to calculate the fatigue damage.

Taking the details in turn, the effective length of the 8 mm fillet weld between the tube and end plate is 334 mm. The force /mm is:

$$\frac{187}{334} = 0.56 \text{ kN/mm}$$

The throat thickness is 5.7 mm. The fatigue direct stress is:

$$\sigma_r = \frac{0.56 \times 10^3}{5.7} = 105 \text{ MPa. This stress factored as described corresponds to } S_k$$

in the curve in Figure 2. The weld detail class is 40, described as "circular structural hollow section fillet welded end to end with an intermediate plate" in Table 8.6 of Part 1-9.

An example of the steps in the summation are given in the Table 1 for 10 intervals.

Index	$\log_{10} N_g \text{ int}$	n_i	$n_{Ei} = n_i + 1 - n_i$	$\gamma_{MF} \gamma_{FF} \Delta S$	$\Delta \sigma_i$	N_{Ri}	n_{Ei}/N_{Ri}	cum n_{Ei}/N_{Ri}
0	0	1	1	116	116	83000	0.0	0.0
1	0.68	4	3	104	110	97200	0.0	0.0
2	1.36	22	18	90.0	96.8	141000	0.0	0.0
3	2.04	109	87	77.9	83.9	216000	0.0	0.0
4	2.72	526	417	66.8	72.4	338000	0.001	0.002
5	3.40	2520	1997	56.5	61.6	546000	0.004	0.005
6	4.08	12100	9567	46.9	51.7	926000	0.010	0.016
7	4.76	57900	45831	38.1	42.5	1660000	0.028	0.043
8	5.44	277000	219568	30.1	34.1	3230000	0.068	0.111
9	6.12	1330000	1051898	22.8	26.4	8660000	0.121	0.233
10	6.80	6370000	5039407	16.2	19.5	39700000	0.127	0.356

Table 1: Calculation steps for 10 intervals

Using 100 intervals gives cumulative damage of 0.320.

For the tube to end plate weld, the damage summation equals 0.32 < 1.0 so the detail is satisfactory.

The second detail is the double-sided fillet weld between the end plate and the spade-end. The effective length of the weld between the tube and end plate is 388 mm. The force /mm is:

$$\frac{187}{388} = 0.48 \text{ kN/mm}$$

The fatigue direct stress is: $\sigma_r = \frac{0.48 \times 10^3}{5.7} = 85.2 \text{ MPa}$. This stress when

factored corresponds to S_k in the curve in Figure 2. The weld detail class is 36*, described as "root failure in partial penetration Tee-butt joints or fillet welded joint ..." in, Table 8.5 of Part 1-9.

For the spade end to end plate weld, the damage summation equals 0.296 < 1.0 so the detail is satisfactory.

Conclusion

The foregoing examples indicate that for a bracing end connection, the predicted fatigue damage according to EC3 Part 1-9 indicates a fatigue life in excess of the normal 50 year design life of a building. This supports the inclusion of clause 2.4.3 in BS 5950:2000 and suggests that following the historical practice in the UK of not carrying out fatigue checks on bracing in conventional buildings is justified when designing to BS EN 1993-1-1 and Part 1-9.

References

- 1 Introduction to fatigue design to BS EN 1993-1-1, New Steel Construction, September 2018

The design of tee sections in bending

David Brown of the SCI looks at the lateral torsional buckling resistance of tee sections, considering the rules in BS 5950 and BS EN 1993-1-1

A tee section? In bending?

A tee section seems an unlikely choice for a member in bending, but judging by the calls to SCI's Advisory Desk, designers do wish (or are perhaps required) to use them. Normally, a tee might be used as a tie between floor beams. The vertical web fits between floor units and the flange sits just below the units, making little impact on an uninterrupted soffit. Before hollow section trusses became popular, tees would have been a good choice for the chords of roof trusses. The web of the tee (if cut from a UB section) provides enough room to connect the angle internal members, either by bolting or welding.

This article considers the alternative ways to design a tee section in both BS 5950 and BS EN 1993-1-1, illustrated with a worked example, so that designers have a resource if faced with the challenge of an unrestrained tee in bending.

BS 5950 guidance

The verification of a tee is covered in Section B.2.8, which provides rules to calculate the equivalent slenderness for lateral torsional buckling (LTB). The first point to note is that guidance is given on when LTB should be considered, and when not. To avoid confusion with Eurocode terminology, the axis on the web centreline will be referred to as the minor axis and the perpendicular axis, the major axis.

In B.2.8.2 a), the Standard advises that if $I_{\text{major}} = I_{\text{minor}}$ LTB does not occur and λ_{LT} is zero. The same applies to doubly-symmetrical sections where there is no reason for the section to buckle in the minor axis.

The reverse is true for tees cut from a UB – major axis inertia is larger than the minor axis inertia and LTB is possible.

Part b) of the clause notes that "if $I_{\text{minor}} > I_{\text{major}}$ LTB occurs about the major axis and λ_{LT} is given by: $\lambda_{\text{LT}} = 2.8 \left(\frac{\beta_w L_e B}{T^2} \right)^{0.5}$ " where B is the flange breadth and T is the flange thickness. Many tees will fall into this category – notably those cut from UC sections where the web is short and the flange is wide and thick. A simply supported tee section with $I_{\text{minor}} > I_{\text{major}}$ loaded so as to put a short unrestrained stem in compression will buckle by twisting to reduce the compression in the stem.

This clause may lead to some significant confusion, because the expression for λ_{LT} for a tee is the same as the equivalent expression for a plate bent about its major axis, given in clause B.2.7. The expression is based on the St Venant torsional stiffness of the flange only; the stem of the tee and any warping stiffness are ignored, hence the similarity with the expression for buckling of a flat plate.

Finally, part c) of the clause describes when $I_{\text{major}} > I_{\text{minor}}$ (the common situation for tees cut from UB) and provides the familiar (for designers of a certain age!) expression: $\lambda_{\text{LT}} = u v \lambda \sqrt{\beta_w}$

The clause goes on to provide expressions for the relevant section properties needed to evaluate λ_{LT} , but designers will mostly obtain these from section property tables. In this case, the warping stiffness of the section is included in the determination of λ_{LT} .

BS EN 1993-1-1 guidance

For tees, there is no change from the normal procedure. To calculate the non-dimensional slenderness $\bar{\lambda}_{\text{LT}}$ the elastic critical buckling moment, M_{cr} is needed. This challenge is conveniently addressed by using software.

Verification methods

In the particular example chosen, the tee is cut from a UB, and thus has a relatively long web. Classification to either Standard leads to the conclusion that the tee is slender (BS 5950) or class 4 (BS EN 1993-1-1).

Two approaches are then possible in both Standards. Either the design stress can be reduced until the section becomes Semi-compact/Class 3, or an effective section can be determined by neglecting the ineffective parts of the cross-section. This latter approach becomes more involved in the Eurocode, because the effective section depends on the stress ratio in the web, which depends on the position of the neutral axis, which moves as the effective section reduces – so an iterative process is needed. BS 5950 is more straightforward as uniform stress in the web is assumed.

Worked example

The tee is a $152 \times 229 \times 30$, in S355, with a buckling length of 4 m. The applied moment is in the plane of the web about the major axis and the web is in compression. The section is shown in Figure 1.

Method 1 – BS 5950 reduced design stress

From look-up tables, d/t for the web = 28

From Table 11, the Class 3 limit is 18ϵ , and as $\epsilon = 0.88$, the limit is 15.84. The section is therefore slender.

Clause 3.6.5 allows the use of a reduced design stress, p_{yr} given by:

$$p_{\text{yr}} = \left(\frac{15.84}{28} \right)^2 \times 355 = 114 \text{ N/mm}^2$$

Various section properties are needed from section tables:

minor axis radius of gyration,

$$r_{\text{yy}} = 32.3 \text{ mm}$$

buckling parameter, $u = 0.648$

monosymmetry index, $\psi = -0.746$ (negative as the flange is in tension)

elastic modulus, $Z = 111 \text{ cm}^3$

plastic modulus, $S = 199 \text{ cm}^3$

With some careful spreadsheet work:

$$v = 1.05$$

$$w = 0.00449 \text{ (includes the warping constant)}$$

$$\beta_w = 111 / 199 = 0.558$$

$$\lambda = 4000 / 32.3 = 123.8$$

$$\text{Then } \lambda_{\text{LT}} = 0.648 \times 1.05 \times 123.8 \times \sqrt{0.558} = 62.9$$

The bending strength can then be calculated from B.2.1, with the result that

$$p_{\text{b}} = 105 \text{ N/mm}^2$$

The buckling resistance moment

$$M_{\text{b}} = 105 \times 111 \times 10^{-3} = 11.7 \text{ kNm}$$

Method 2 – BS 5950 effective section method

Given that the section is slender, an effective section may be calculated. Clause 3.6.2.2 prescribes that the effective width of a class 4 slender outstand should be taken as equal to the class 3 limiting value (18ϵ , as above).

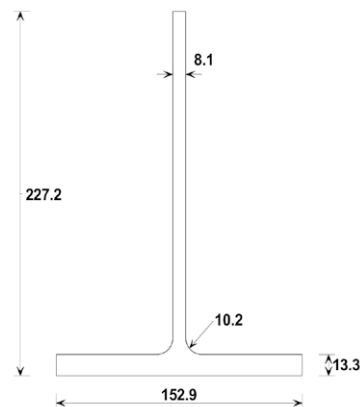


Figure 1: Tee section dimensions

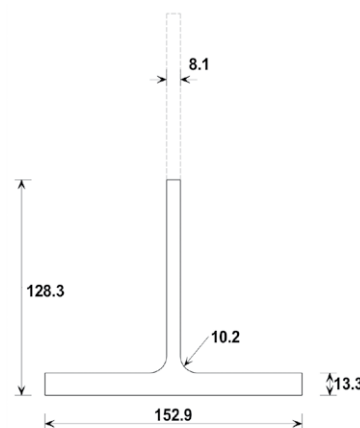


Figure 2: BS 5950 effective section

The overall depth of the effective section is therefore $18 \times 0.88 \times 8.1 = 128.3$ mm. The dimensions of the effective section are shown in Figure 2. Calculations are required to determine the position of the neutral axis (accounting for the root radii if doing a 'proper' job!), and calculating the effective elastic modulus of the section. The effective elastic modulus is calculated as 36.3 cm^3 .

$$\beta_w = \frac{36.3}{199} = 0.18$$

$$\text{Then } \lambda_{LT} = 0.648 \times 1.05 \times 123.8 \times \sqrt{0.18} = 35.7$$

Following the same process from B.2.1, the bending strength,

$$p_b = 339 \text{ N/mm}^2$$

$$\text{The buckling resistance moment } M_b = 339 \times 36.3 \times 10^{-3} = 12.3 \text{ kNm}$$

Method 3 – BS EN reduced stress method

The ratio for local buckling is defined differently in the Eurocode, which species c/t as the dimensions of the outstand, not overall depth.

$$c/t = \frac{(227.2 - 13.3 - 10.2)}{8.1} = 25.2$$

The limiting value depends on the stress ratio between the stress at the tip of the web, and at the root radius (refer to Table 5.2 in BS EN 1993-1-1). To evaluate the limit, BS EN 1993-1-5 must be consulted to calculate the buckling factor, k_σ .

If the neutral axis is at 58.4 mm from the face of the flange (from section property tables), the stress ratio may be calculated from the dimensions shown in Figure 3.

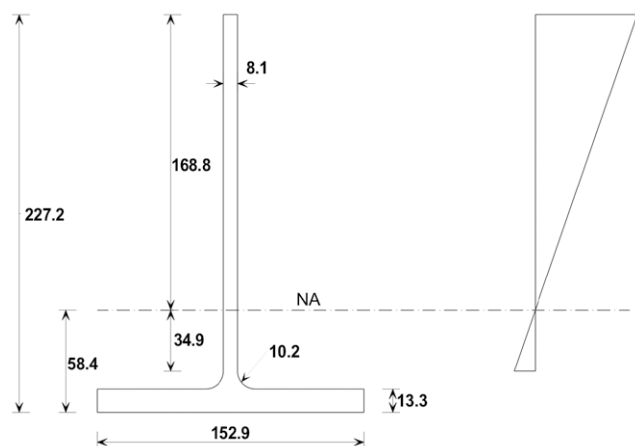


Figure 3: Elastic stresses in the web of the gross section

$$\psi = \frac{-34.9}{168.8} = -0.207$$

From Table 4.2 of BS EN 1993-1-5, then

$$k_\sigma = 0.57 - 0.21\psi + 0.07\psi^2$$

$$k_\sigma = 0.57 - 0.21 \times (-0.207) + 0.07 \times (-0.207)^2 = 0.616$$

Back in BS EN 1993-1-1 Table 5.2,

$$\text{the limit is } 21\sqrt{k_\sigma} = 21 \times 0.81 \times \sqrt{0.616} = 13.3$$

$25.2 > 13.3$, so the section is class 4 (not surprisingly, given the BS 5950 classification)

To ensure the section remains class 3, the reduced design strength

$$\text{is given by } 235 / \left(\frac{25.2}{21 \times \sqrt{0.616}} \right)^2 = 100.5 \text{ N/mm}^2$$

M_{cr} must be calculated, using the gross properties. *Ltbeam* is a convenient software to use. With a UDL causing compression on the web, $M_{cr} = 67$ kNm. Verification then proceeds in the usual way, using the general case of clause 6.3.2.2. A tee section is taken to be an "other cross section" in Table 6.4. The intermediate values are therefore:

$$\bar{\lambda}_{LT} = 0.41$$

$$\alpha_{LT} = 0.76$$

$$\phi_{LT} = 0.66$$

$$\chi_{LT} = 0.84$$

$$\text{and finally } M_{brd} = 9.5 \text{ kNm}$$

Method 4 – BS EN effective section method

Having found the section is class 4, the effective length of the web may be determined from BS EN 1993-1-5.

If $k_\sigma = 0.616$ then from clause 4.4(2)

$$\bar{\lambda}_p = \frac{b/t}{28.4\epsilon\sqrt{k_\sigma}} = \frac{25.2}{28.4 \times 0.81 \times \sqrt{0.616}} = 1.39$$

Because $\bar{\lambda}_p > 0.748$ then

$$\rho = \frac{\bar{\lambda}_p - 0.188}{\bar{\lambda}_p^2} = \frac{1.39 - 0.188}{1.39^2} = 0.622$$

The effective length of the web from the neutral axis is therefore $0.622 \times 168.8 = 105$ mm and the overall depth of the effective section is now 163.7 mm.

This change of section means that the original assumptions about c/t ratio, position of neutral axis etc are now invalid. The process must be repeated (by spreadsheet preferably!) until a final solution is found. A final solution is found when there is no further reduction needed to the web (i.e. all the reduced section is effective). This happens when $\rho = 1$ (no reduction), which, with reference to BS EN 1993-1-5, happens when $\bar{\lambda}_p = 0.748$

Probably, there would be a neat way to determine this point by calculation, but it is easy to complete a number of cycles to discover the point when the entire reduced section becomes effective. The final section, with an overall depth of 130 mm, is shown in Figure 4. The Eurocode effective section appears reassuringly similar to that according to BS 5950, in Figure 2.

Having found the final section, the section properties can be determined and the resistance determined in the normal way, as Method 3. The intermediate values are:

$$W_{el} = 37.3 \text{ cm}^3$$

$$\bar{\lambda}_{LT} = 0.44$$

$$\alpha_{LT} = 0.76$$

$$\phi_{LT} = 0.69$$

$$\chi_{LT} = 0.82$$

$$\text{and finally } M_{brd} = 10.8 \text{ kNm}$$

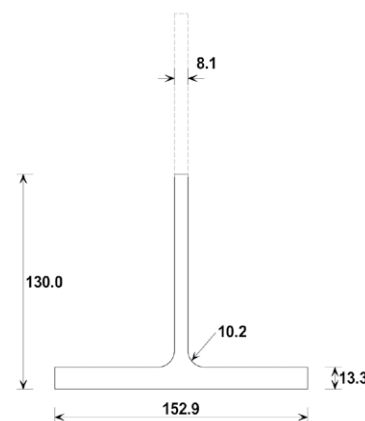


Figure 4: EN 1993 effective section

Summary

The various resistances are shown below:

BS 5950 reduced design strength	11.3 kNm
BS 5950 effective section	12.3 kNm
BS EN 1993-1-1 reduced design strength	9.5 kNm
BS EN 1993-1-1 effective section	10.8 kNm

Note that according to BS 5950, the maximum moment should be limited to M_b/m_{LT} , so the BS 5950 values above should be increased by $1/0.925$ to provide a proper comparison. The shape of the bending moment diagram – due to a UDL – is already included in the Eurocode resistances by virtue of the M_{cr} value.

Conclusions

Firstly, it is not easy to calculate the correct resistance. It took some time and the assistance of two colleagues at SCI to reach a consensus. The Eurocode approach has the benefit of software to calculate M_{cr} , but the easier solution (method 3, reduced design strength) is conservative. The less conservative method 4, effective section, is painful because of the loops required to calculate the effective section.

The second observation is that perhaps the guidance in BS 5950 could be clearer.

The final observation is that tees have their place - but preferably not as unrestrained members in bending.

Cross-braced lateral load-resisting systems

Cross bracing is a traditional means of providing lateral stability to structures. Richard Henderson of the SCI discusses some of the features of this structural system.

As structural engineers of a certain age will recall from their student days a cross-braced panel is a statically indeterminate (or hyperstatic) structural system: the forces in the members cannot be determined simply by invoking equilibrium at the joints. Determining the forces used to be an exercise in the application of virtual work to structural problems.

When cross bracing is used to resist lateral loads, the bracing members are usually designed as tension only and the designer assumes that the element which forms the compression member buckles elastically as the frame deforms so as to shorten the relevant diagonal. This approach is favoured when analysing and designing structures by hand as determining the buckling resistance of the member is avoided. Crossed flats were traditionally used for this purpose although angle bracing could be used so the bracing members had some out of plane stiffness to make handling easier. Cautionary tales regarding finishes being pushed off by bowing bracing are told, leading to the adoption of different bracing arrangements.

Flat bar bracing

A flat bar tension only bracing member in a 4 m × 6 m pin-jointed braced panel (say a 130 mm × 10 mm flat), bolted to the opposing diagonal member at the centre, has a system length of $\sqrt{13}$ m, assuming the tension diagonal provides a point of restraint at the centre connection. (For a detailed assessment see BS EN 1993-2 Annex D). The out of plane second moment of area is $1.083 \times 10^4 \text{ mm}^4$ giving an Euler buckling load:

$$N_{cr} = \frac{\pi^2 \times 210 \times 1.0833 \times 10^4}{13 \times 10^6} = 1.73 \text{ kN}$$

The buckling resistance of the member $N_{b,Rd}$ is very close to the Euler load because of the high out of plane slenderness and has a value of 1.69 kN, assuming S355 material. A compression force of this magnitude is unlikely to have any effect on a bracing connection designed for a tension force of 450 kN and is usually safely ignored.

An estimate of the bow in the compression member which is making no contribution to the lateral resistance of the braced panel can be made if the panel members are known, assuming the member buckles into a circular arc. As an example, assume 203 UC 71 columns and a 406 × 178 UB 54 beam framing the 130 × 10 flat cross bracing (Figure 1), with a horizontal design load of 374 kN applied to the braced panel.

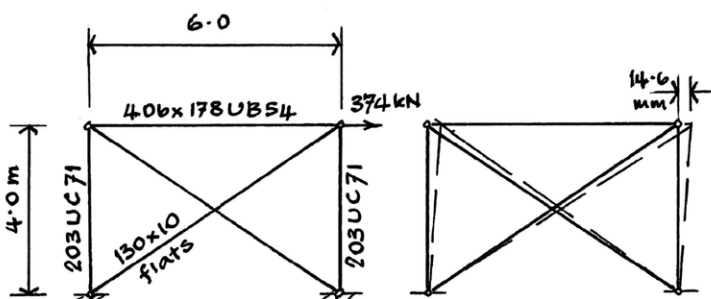


Figure 1: Braced panel

The horizontal displacement of the top of the panel relative to the bottom is 16.2 mm or 14.6 mm depending on at which end of the beam the force is applied and the displacement calculated. The extension of the bracing is about 12.1 mm (taking the smaller displacement). If the shortening of the opposing diagonal is taken as the same value, the bow is about 94 mm (neglecting the elastic shortening of the bracing member under the axial load). If the flat is unrestrained in the middle, the bow is about 180 mm. Clearly, such a bow could be sufficient to push dry lining off a wall concealing the braced panel. The low Euler load indicates clearly that the member buckles elastically and will behave satisfactorily when the loads are reversed.

An elastic stick finite element analysis that includes all the members without somehow allowing for the buckling behaviour of the bracing will produce a diagonal load in the compression member which corresponds to its axial stiffness. In such an analysis, the tension and compression diagonals share the load and carry a force which is close to half the force in the member assuming tension-only.

Tubes used as tension only bracing

An alternative form of bracing member consisting of RHS tubes, also assumed to behave as tension-only, is sometimes adopted. Consider 90 × 50 × 5 RHS tubes with centrelines in the same plane with a welded joint in the middle. Assume for the purpose of this example that the middle joint is pinned and behaves in a similar way to the crossed flats in providing a point of restraint in the middle of the compression member. The minor axis buckling resistance of the RHS for a length of $\sqrt{13}$ m is 71.6 kN by calculation. The compression member therefore carries a force of at least 71.6 kN which the connections must be able to sustain. The maximum theoretical load on the connection is equal to the Euler load and is equal to 78.4 kN, about 9.5% higher. If the connection (perhaps a gusset) is designed for tension only, it is possible that a load equal to the compression resistance is sufficient to deform the gusset permanently, compromising its ability to resist tension when the bracing load is reversed.

The amplified bow in the bracing member that corresponds to the buckling resistance can be found from back calculation. The assumed initial bow e_0 is given by:

$$e_0 = \frac{W}{A} \alpha (\bar{\lambda} - 0.2)$$

where $\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\frac{1270 \times 355}{78420}} = 2.4$, and the

imperfection factor for an RHS = 0.21.

Substituting values in the formula for the initial bow gives:

$$e_0 = \frac{19.7 \times 10^3}{1270} \times 0.21 \times (2.4 - 0.2) = 7.16 \text{ mm}$$

The amplified bow at failure is

$$\frac{N_{cr}}{N_{cr} - N_{b,Rd}} e_0 = 11.48 \times 7.16 \approx 82 \text{ mm}$$

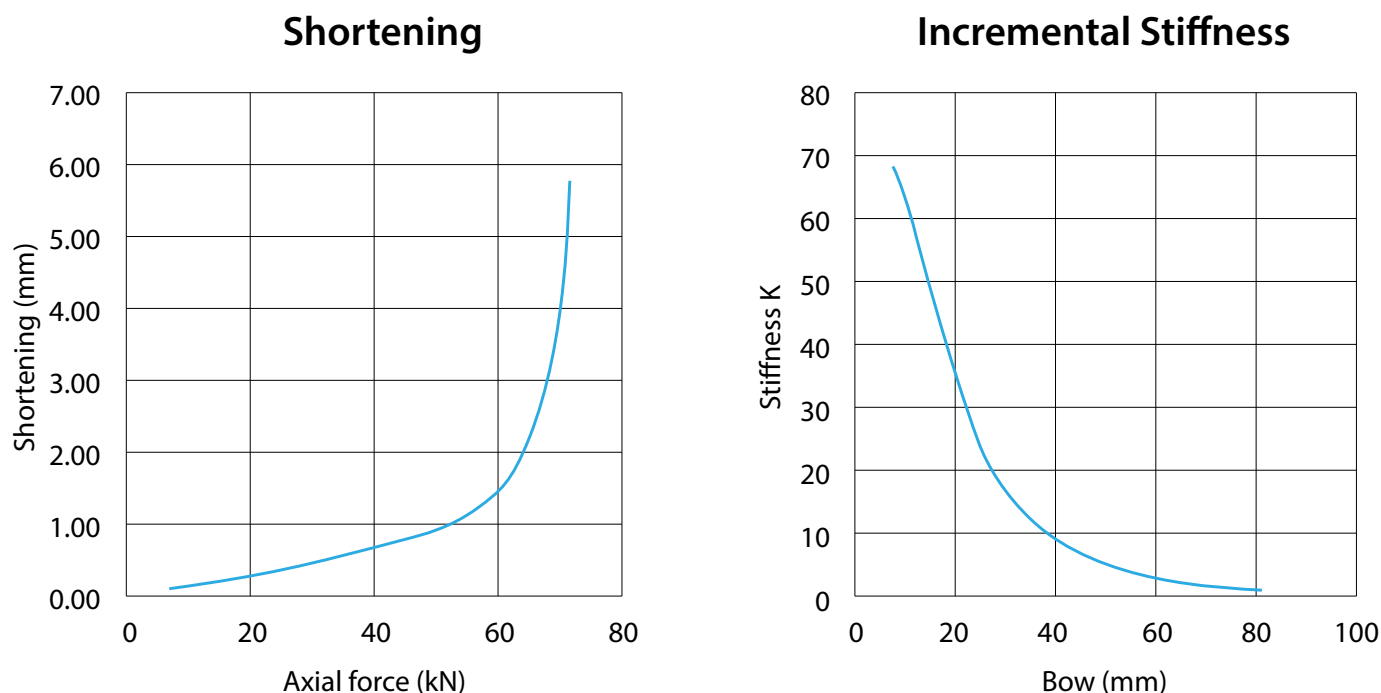


Figure 2: Member shortening and incremental stiffness

This is the bow at which the extreme fibre at the point of maximum bow (and bending moment) reaches yield stress due to combined axial load and bending. The bow is about 15% less than that in the flat bar. As the frame deflects and load on the member is increased, the bow increases, the member shortens more and more quickly and the stiffness of the compression member decreases as shown in Figure 2. The member reaches its buckling load as the frame reaches its maximum sway deflection of 14.6 mm.

Column shortening

If a cross-braced panel with bracing that is intended to behave as tension-only has significant axial loads in the columns, the bracing will develop axial loads which may confuse the unwary. An elastic stick finite element analysis which includes all the elements in the model with pinned connections and which makes no provision for members intended to buckle when in compression, will exhibit compression forces in the bracing and a tension force in the beams: see Figure 3. The forces may or may not be sufficient to cause the bracing members to buckle, depending on the magnitude of the applied forces and the bracing section chosen.

If the braced panel is modelled with pinned joints and only the tension element present and if only vertical loads are applied, no axial forces will be developed in the bracing member or beams. The braced panel will deflect sideways however, to accommodate the bracing member which remains at its original length.

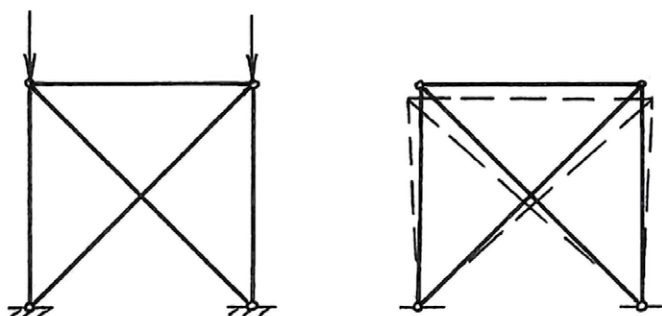


Figure 3: Deflection under vertical loads

Lateral stiffness

It is advantageous to mobilise both tension-only bracing members in a cross-braced panel if this can be achieved, because the increased stiffness is beneficial to the overall stability of the building. The contribution of the bracing members to the lateral stiffness is of course doubled and the magnitude of the α_{cr} value for the building increased, thereby reducing any amplifier on the lateral loads. A cross bracing system formed of rods, perhaps adopted for architectural reasons, can be pre-tensioned to prevent the rod forming the compression diagonal from going slack. In this case, the bracing members in both diagonals will be effective as the tension force in the member in the shortening diagonal will be reduced as the bracing resists a lateral load. There are proprietary systems of rods, rod-ends, turnbuckles and connecting rings which are designed to achieve this effect¹.

Tensioned bracing is more difficult to achieve when the bracing members are a different geometry from rods. In the past it has been standard practice in some drawing offices to detail the holes in cross bracing members such that the length of the diagonal is 5 mm "short". This required the erection team to lean the columns when making the connections for the first bracing member to be erected. Installing the second member was much more difficult as it involved tensioning the first diagonal so as to shorten the opposing diagonal by enough to make the final connection.

Conclusion

Tension-only bracing members provide a simple means of resisting lateral loads on a structure but certain features of the behaviour of the bracing need to be considered:

- 1) The slack member of flat bar cross bracing can bow significantly which could possibly damage finishes.
- 2) If using tubes as cross bracing, the connections must be capable of resisting a compression force at least equal to the buckling resistance of the member.
- 3) A simple stick finite element analysis model of a frame with cross-bracing will develop compression forces in both bracing members unless steps are taken in the analysis to avoid this.
- 4) Mobilising both bracing members (eg by pre-tensioning) increases the α_{cr} value of the frame and is therefore beneficial.

1. Round bar cross bracing, p21 NSC, September 2015

Connection design in trusses

A general article about steel trusses which touched on choice of members and their orientation but did not go into detail about designing connections was published in 2017¹. In the present article Richard Henderson of the SCI illustrates the implications of such choices on the connection design.

Introduction

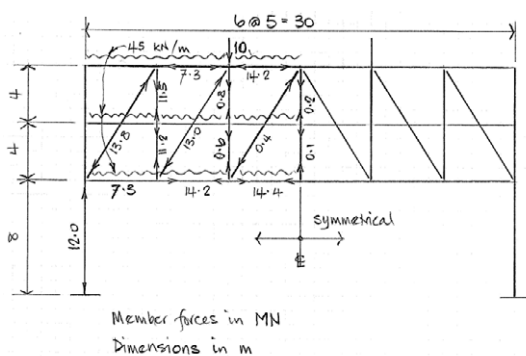
The selection of members and their orientation and the impact on the design of connections in a truss is best illustrated with an example. The arrangement of the truss, the magnitude of the forces and the orientation of the members all have an impact on the form of the connections. A fundamental part of achieving an efficient joint design is establishing an understanding of the flow of forces through the joint. This is only possible if the forces provided for the design of the joint are in equilibrium. If envelope forces are provided, this compromises the designer's ability to develop an efficient connection design. In what follows connections between open section members will be considered.

As an example, consider a transfer truss spanning 30 m supporting two columns at third points, each carrying 10 MN from floors above. The truss is divided into three bays of 10 m width by the columns. The building storey height is 4.0 m and each floor in the truss will carry a uniform load of 45 kN/m. The chords are restrained out of plane by floor beams perpendicular to the plane of the truss.

Truss arrangement

An early decision is what the depth of the truss should be. The maximum bending moment is 100 MNm from the columns and about 5 MNm from each floor. If the truss is one storey deep (ie a span to depth ratio of 7.5), the maximum chord force is 27.5 MN which exceeds the axial resistance of the largest UC section. In this example, a two-storey truss is chosen, giving a maximum chord force of about 14.4 MN which can be carried by a UC. The truss can be conveniently divided into 5 m panel widths. An N-frame or Pratt truss has shorter vertical members in compression and longer diagonal members in tension. The connections in the tension members are likely to prove the most difficult to detail and the tension forces in the bracing could be reduced by orienting the bracing so that the diagonal members are in compression and the verticals in tension (Figure 1).

Figure 1:
Truss arrangement
diagonals in
compression

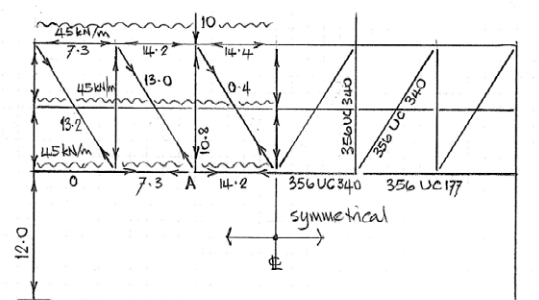


It can be seen from Figure 1 that the length of the bottom chord carrying a force above 14 MN is more than 20 m long and will need a tension splice for transportation. Adopting a conventional N-frame is therefore considered to be preferable as the necessary splices can be located in elements with lower forces.

A truss with a single storey depth could be shop-fabricated and transported to site in three pieces with two bolted site splices. Erection time would be reduced but crane lifting capacity and transportation would

need to be considered if this option is contemplated. The truss arrangement chosen, member forces and some member sizes are shown in Figure 2.

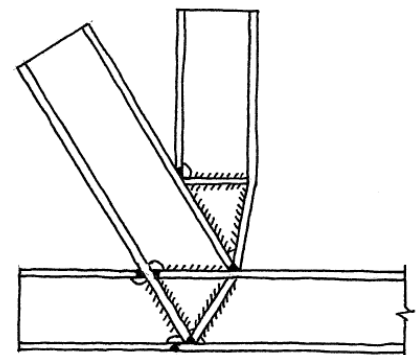
Figure 2:
Truss arrangement
diagonals in
tension



Example connection design – orientation of members

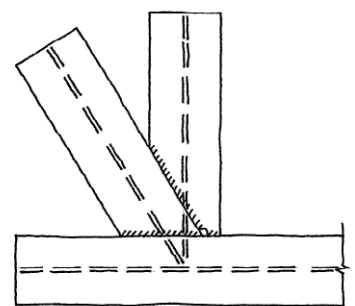
Consider the joint at point A at the base of the column carrying 10 MN. The bottom chord member could be detailed as one fabricated assembly with a joint at each end to connect to the column, diagonal brace and the continuing chord member. At this joint, there is a tension in the central section of bottom chord of about 14.2 MN and a tension diagonal carrying about 13 MN. The chord member carries about 40% of the tension in each flange and 20% in the web. A conventional orientation of the members might be considered with the webs vertical as in Figure 3.

Figure 3:
Bottom chord joint -
webs vertical



A path for transferring the flange forces from the chord to the bracing member is necessary (because the forces are obviously too large to transfer through the webs) and an additional load bearing stiffener is necessary to carry the resultant force at the change in direction. As the forces are in tension, full strength welds would be required. The butt welds between the flanges are substantial and require cope holes through the web to achieve

Figure 4:
Bottom chord joint -
flanges vertical



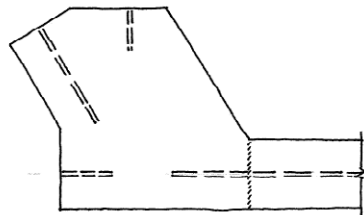
them. The webs would need to be checked for shear as well as axial load in the joint zone. This arrangement is not favoured.

Rotating the members so their flanges are vertical (Figure 4) provides a more direct path for the flange tension forces.

The connections between the members in the node can be made by butt welds between the edges of the flanges. The flow of force through the joint is smoother but the web force still needs to be transferred, and the junction where the webs of the three members come together is complicated.

A refinement of this arrangement, using two plates to form the node, separated by intermittent webs is the favoured solution (Figure 5). The plates toward the centre of the joint are wide enough to carry half the chord force so a web is only required close to the connecting member to transfer the web force into the plates. The plates are butt welded to the chord flanges and the tension diagonal is connected using a bolted splice. The column member is connected using a bearing splice.

Figure 5:
Bottom chord joint –
splice plates



Joint design

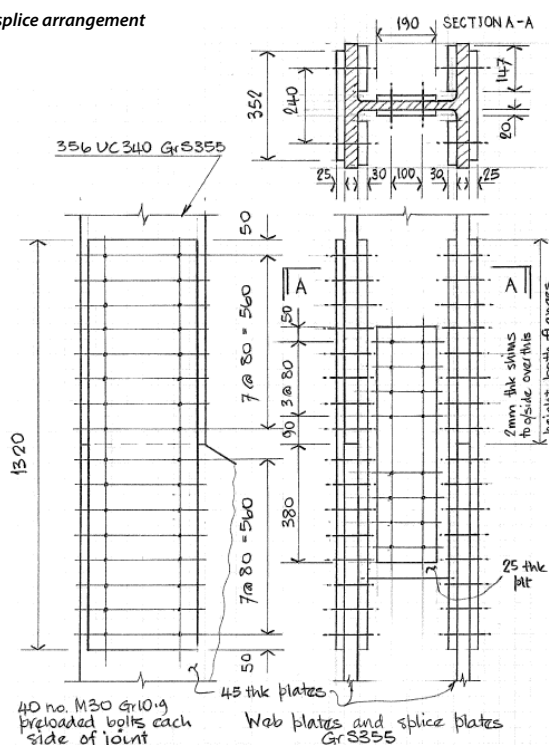
The tension splice in the bracing member will be effected using M30 preloaded bolts of category B in double shear, of grade 10.9. The slip resistance assumed for design is for a friction coefficient $\mu = 0.5$ and is 357 kN.

The member is a 356 UC 340 with 42.9 mm thick flanges and 26.6 mm thick web and has an area of 433 cm². The area of one flange is 40% of the total and carries 5.2 MN in tension. The number of bolts required is indicated in Table 2:

	Force (MN)	No of Bolts	Adopt
Flange	5.2	14.6	16 bolts
Web	2.6	7.3	8 bolts

The flange splice plates are chosen to provide the same area of metal as the flange with half the area on each side to balance the force on each shear plane in a bolt. The splice arrangement is shown in Figure 6. All bolts in the truss will be M30 grade 10.9 preloaded assemblies, category B.

Figure 6: Bolted splice arrangement



The inside face of the plates in the node are arranged to line through with the inside face of the element flanges. Externally, shims are provided to reduce the difference in thickness to less than 1 mm.

The bearing splice in the column member must be designed for 25% of the maximum compression ie 2.7 MN. Dividing by the double shear resistance gives 7.6 bolts and four bolts will be used in each flange.

The tension connection for the continuation of the bottom chord will be detailed in a similar way to the tension diagonal with ten bolts in the flanges and six bolts in the web. The difference in flange thickness in this part of the joint is 21 mm and is achieved with two shims of 15 mm and 6 mm thickness.

The connection between the bottom chord and the node plates is required to transfer 14.2 MN in tension. The node plates will be butt welded to the bottom chord member. The force in the web will be transferred by welds to the node plates; either fillet welds or partial penetration butt welds can be used. This can be achieved either by stripping both flanges off the member to allow the web to project between the node plates or by butt welding an extension plate to the web. Short web plates are required between the node plates at each bolted connection to receive the web force and transfer it into the node plates through fillet welds.

The node plate geometry is such that the stress in the plates reduces rapidly away from the interface with the connected members. The resistance to compression from the vertical column must also be considered. A buckling check of the compression force in the node plates should be carried out. The final joint arrangement is shown in Figure 7.

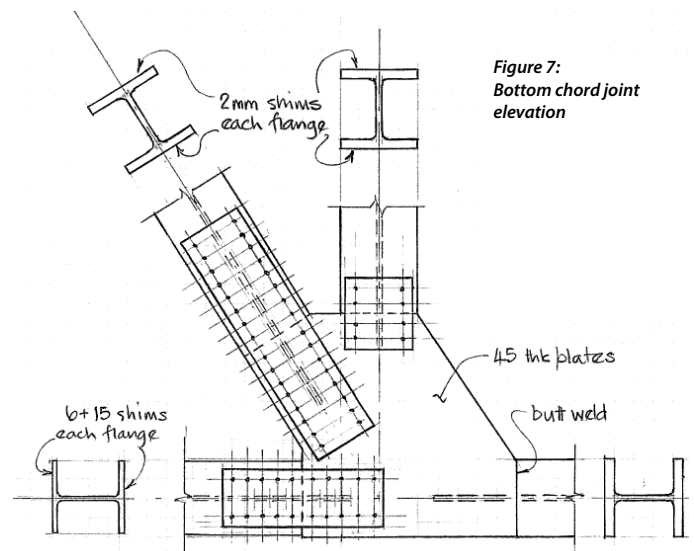


Figure 7:
Bottom chord joint
elevation

Conclusions

1. Early consideration of the form of members in the truss (rolled or fabricated) may influence the depth adopted. Transportation, craneage, erection and the proportion of shop fabrication also influence the truss arrangement.
2. The flow of forces through the joint is clear if the forces are in equilibrium. The orientation of members (webs vertical or horizontal) affects the need for and nature of welds required to transfer the forces.
3. Facilitating the flow of forces between the element flanges results in a joint arrangement where member stubs are not welded together but plates are provided, aligned with the member flanges to carry the forces through the joint which reduces the stiffening required and the amount of welding.
4. Fewer bolts would result if non preloaded grade 10.9 bolts were adopted (30 on each side of the diagonal member splice instead of 40) but the deflection of the truss would be more difficult to control because of bolt movement in clearance holes and bearing deformation.
5. The double shear resistance of non preloaded M30 grade 10.9 bolts is only 4% greater than grade 8.8 bolts of the same size but this can be enough to produce a smaller number of bolts. Once selected, the bolt grade and size is fixed for the whole truss.

Reference

- 1 Steel construction with trusses, NSC, March 2017

Bolt slip in connections

The effect of bolt slip in truss connections is an issue that is raised with SCI from time to time in various contexts. Richard Henderson discusses some of the issues.

Introduction

The deflection of a truss can be estimated using various analytical methods and often a stick finite element (FE) package will be used to determine the member forces and the deflections under the different load cases. The calculated deflection depends on the assumptions made in the analysis about the nature of the joints – whether pinned or rigid.

Truss Joint types

In BS EN 1993-1-8, three categories of bolted connection loaded in shear are identified:

- Category A: bearing connections where the bolts act in shear and bearing;

Connections made with preloaded bolts:

- Category B: slip-resistant at serviceability limit state;
- Category C: slip-resistant at ultimate limit state.

Connections in category B must also be designed for shear and bearing in the ultimate limit state and Category C for bearing and net area. Fewer bolts will be required in Category B connections than in Category C ones.

SCI recommends adopting joints made with preloaded bolts where members are spliced and deflection is of concern because this allows the deflection of a truss to be better controlled. Category B joints are usually sufficient but Category C joints may be specified in special cases (eg with oversize or slotted holes). In theory, once the joints are made, the subsequent deflection of the structure is due only to the elastic deformation of the members.

Predicting deflections in trusses

As discussed in the introduction, an FE model of a truss will deliver the deflections of the structure as well as the member forces for a given load case. The actual deflection of a truss made with Category A bolted joints may well be greater than the predicted deflection, because the joints may slip when the load comes onto the structure and the bolts take up their loaded position. The deflection will be more significant if holes are oversize or slotted. This effect may be predicted by using virtual work methods which assume a pin-jointed model and adding an allowance for the slip at each bolted connection to the extension of the member due to the internal forces. This can be illustrated by example.

Example 1

Consider a two element pin-jointed bracket connected to rigid supports as shown in Figure 1. Estimate the total deflection if there is a 2 mm slip in each bolted connection.

Considering the elements separately for displacements that are small relative to the lengths of the members, if there is a change in length in the elements of 2 mm due to bolt slip, the vertical deflection in millimetres resulting from the extension of the diagonal is $2/\sin\theta$ and $2/\tan\theta$ from shortening of the horizontal member. The total deflection is therefore $2 \times (1/\sin\theta + 1/\tan\theta) = 4.8$ mm for $\theta = 45^\circ$.

The same calculation by virtual work is given in Table 1.

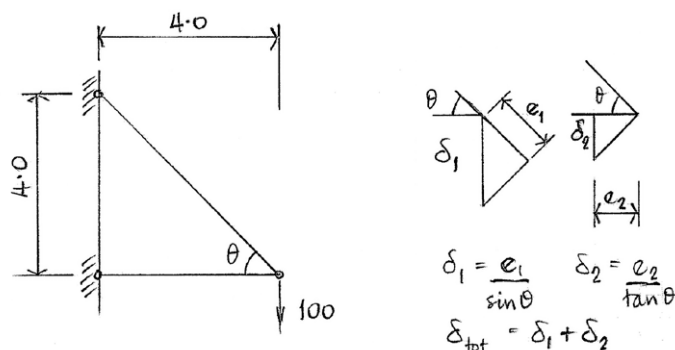


Figure 1: Bracket arrangement

Element		Diagonal	Strut	Total (mm)
Area (mm ²)	A	470	667	
Length (m)	L	$4\sqrt{2}$	4	
Member forces (kN)	p_1	$100\sqrt{2}$	100	
Member forces due to unit load	p_2	$\sqrt{2}$	1	
Member flexibility (mm/kN)	L/EA	0.0573	0.0286	
Member deformation (mm)	$p_1 L/EA$	8.1	2.9	
Deflection due to member deformation (mm)	$p_2 p_1 L/EA$	11.5	2.9	14.4
Slip (mm)	s	2.0	2.0	
Deflection due to slip (mm)	$p_2 s$	2.8	2.0	4.8
Total deflection: $\sum p_2 (p_1 L/EA + s)$				19.2

Table 1: Bracket deflection

Both methods give the same deflection due to bolt slip.

Example 2

To illustrate the effect of bolt slip consider a pin jointed Pratt truss (N frame), shown in Figure 2. Member areas are based on a tensile stress of 350 MPa and 150 MPa in compression, with the area limited to a minimum value.

The deflection of the truss centre under the total design load is estimated to be 175 mm (span divided by 230), calculated by virtual work. An FE model gives a deflection of 179 mm. The deflection can be apportioned to 110 mm of bending deflection (deformation of the truss booms) and 65 mm of shear deflection, from the bracing members. Making this distinction is useful if the deflection is to be reduced because the elements making the greatest contribution to the total deflection can be identified.

In estimating the effect of bolt slip, it is assumed that with automated saw and drill lines, the accuracy of holing is such that slip can occur in all holes simultaneously. If all the members are bolted with 2 mm oversize holes and 1 mm of slip is assumed at each end of a member, a total of 2 mm per member, the deflection increases by 43% to about 250 mm. The effect

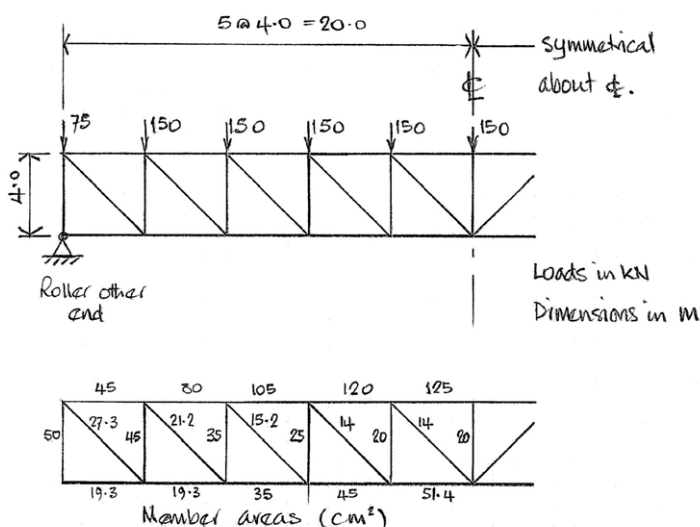


Figure 2: Truss arrangement

on the mid-span deflection of other assumptions about which members experience slip is shown in Table 2. Possible scenarios are 1) that pipe-flange type bearing splices are effected in compression booms with no slip; 2) that both booms are effectively continuous with the bracing members bolted to them and 3) that the truss is shop-welded with bolted splices.

Condition	Deflection (mm)	% increase
No slip	175	-
All members bolted, 1 mm slip in each joint	250	43
No slip in compression boom, 1 mm slip in other joints	220	26
No slip in booms, 1 mm slip in bracing joints	200	14
1 mm slip at 2 bolted splices in booms and diagonals	190	9

Table 2: Effect of bolt slip on deflection – 2mm slip per member

If the most unfavourable assumptions are made about the position of the bolts in their holes a slip of 4 mm at each end of a member is theoretically possible as shown in Figure 3.

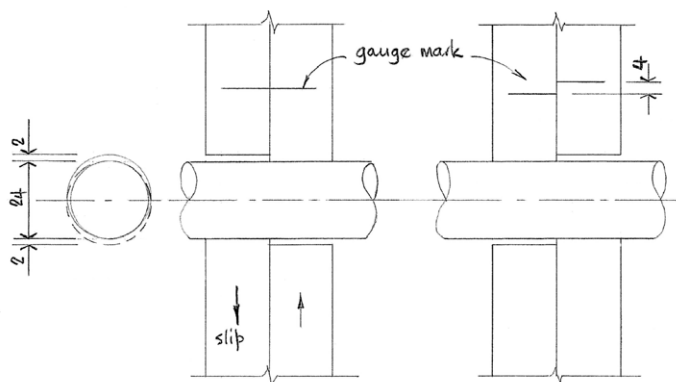


Figure 3: Worst case slip

The corresponding deflections are set out in Table 3. It can be seen that the theoretical increase in the mid-span deflection is very large. This is not

Condition	Deflection (mm)	% increase
No slip	175	-
All members bolted, 8 mm slip in each joint	470	269
No slip in compression boom, 8 mm slip in other joint	350	200
No slip in booms, 8 mm slip in bracing joints	270	55
Slip at 2 bolted splices in booms and diagonals	242	38

Table 3: Effect of bolt slip on deflection – 8 mm slip per member

surprising when the elastic deformations in the compression members are about 3 mm and an average of about 5 mm in the tension members.

A truss designed with joints made with preloaded bolts of Category C where the friction coefficient assumed in design is not achieved may well experience increased deflection in service. However, the magnitude of the increased deflection is uncertain. The potential percentage increases indicated in Tables 1 and 2 are unlikely to be realized for several reasons.

Discussion and conclusion

It is almost certainly not the case that each joint in each member will slip by the same amount, because the force carried per bolt will not be uniform throughout. For example if the number of bolts required in a joint is 6.2, determined by dividing the design load by the bolt resistance, 8 bolts will be provided. This suggests that the possibility of any dynamic effects due to a sudden slip in all the joints is unlikely.

The absolute worst-case increased deflection set out in Table 3 will not occur because in practice the bolts will never be installed in every joint such that the maximum slip can occur. According to the NSSS, the maximum deviation from the intended position of a hole in a group of holes is 2 mm so it is anticipated that there will be some variation in the position of the bolt holes in a group (meaning some bolts will already be in bearing) and reduce the potential slip.

Kulak and others¹ discuss the behaviour of bolted joints and state "High strength bolts are usually placed in holes that are nominally 1/16 in. [1.6 mm] larger than the bolt diameter. Therefore the maximum slip that can occur in a joint is equal to 1/8 in [3.2 mm]. However, field practice has shown that joint movements are rarely as large as 1/8 in. and average less than 1/32 in [0.8 mm]. In many situations the joint will not slip at all under live loads because the joint is often in bearing by the time the bolts are tightened. This might be due to small misalignments inherent to the fabrication process. In addition slip may have occurred under dead load before bolts in the joint were tightened. Generally, slips under live loads are so small that they seldom have a serious effect on the structure".

In practice therefore, the maximum slip at each joint may well be no more than 1 mm.

If further reading is desired, a design guide for single storey steel buildings² published by Arcelor Mittal and others includes a section on estimating deflection due to bolt slip.

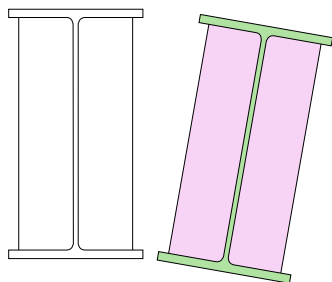
1. Geoffrey L Kulak, John W Fisher, John H Struik, Guide to design criteria for bolted and riveted joints, Second Edition, AISC, 2001
2. Steel buildings in Europe, Single storey steel buildings, Part 5 Detailed design of trusses Section 3.6
https://constructalia.arcelormittal.com/en/news_center/articles/design_guides_steel_buildings_in_europe

Advisory Desk 2019

AD 425

Full depth stiffeners and lateral torsional buckling

The SCI Advisory Desk sometimes receives questions about the potential to use full depth stiffeners to restrain lateral torsional buckling, suggesting that the stiffeners prevent relative movement of the compression and tension flanges. Whilst this is true, lateral torsional buckling is a displacement and twist of the complete section, which stiffeners alone do nothing to prevent. The American Institute of Steel Construction notes that “transverse stiffeners are simply along for the ride” as the sketch indicates.



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AD 426

Bolt head protrusion through nuts and threads in grip lengths

To ensure that bolt threads are fully engaged in the nut, BS EN 1090-2 clause 8.2.2 specifies that the protrusion must be at least one thread pitch. This is because the very end of the bolt may be slightly convex, leading to a reduced resistance if threads are not fully effective.

The same clause specifies the necessary numbers of threads within the grip length (between bolt head and the nut). For non-preloaded bolts, one full thread is required – to ensure the nut can be properly tightened. For preloaded bolts according to BS EN 14399-3 (HR system, generally used in the UK in preference to the HV system) or according to BS EN 14399-10 (HRC system, commonly known as a ‘tension control bolt’), a minimum of four threads within the tensioned length is specified. The reason for the threads in the tensioned length is to encourage ductile behaviour – AD 268 (which related to the BS 5950 requirements) reproduces a figure from Owens and Cheal (Butterworths), showing significantly more elongation when there are more threads in the tensioned length. Incidentally, BS 5950-2 required three and five threads in the tensioned length, for class 8.8 and 10.9 bolts respectively.

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AD 427

Typographical error in P419

A few eagle-eyed readers have noticed a typographical error in SCI publication P419 Brittle fracture: selection of steel sub-grade to BS EN 1993-1-10, within the expression to determine the design crack growth a_d presented in section 3.1.1.

The sign of the fourth term in the expression should be negative and read $-6.3837 \times 10^{-4} t^2$

The typo is repeated in the numerical example in Appendix A section A.2 where the expression is stated again. However, the result of the expression (a design crack depth of 2.26 mm) is correctly stated, having been calculated respecting the correct sign. The tabulated values in the publication have also been determined using the correct expression.

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AD 428:

Draft guidance: lateral and torsional vibration of half-through truss footbridges

Purpose of this guidance

This note alerts designers to the potential susceptibility of narrow half-through footbridges to excitation by pedestrians in a lateral-torsional mode. Eurocodes and UK National Annexes do not currently fully address this mode of vibration, so there is a danger it may be discounted without proper consideration. This gap in the standards has led to the need to retrofit dampers and/or provide additional stiffening to some recently constructed footbridges where excitation occurred due to pedestrians walking eccentric to the deck centreline and, more significantly, from deliberate shaking of the deck.

Affected mode of vibration

Half-through footbridges, without plan bracing to the top chord, often have as their lowest natural mode of vibration a lateral-torsional mode. A typical example is shown in Figure 1. The mode occurs because the open bridge cross-section has a low torsional stiffness with a shear centre below the deck level about which axis the rotation occurs.

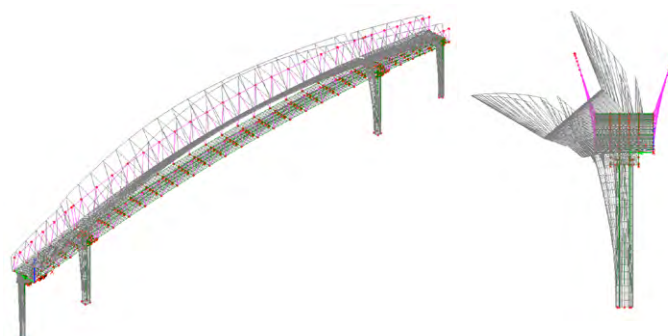


Figure 1 – Lateral and torsional mode of vibration

Current UK design criteria and their interpretation

The criteria for assessing the dynamic behaviour of footbridges are outlined in the following Eurocodes (BS EN) and BSI Published Documents (PD):

- BS EN 1990:2002+A1:2005 as modified by UK National Annex
- BS EN 1991-2:2003 as modified by UK National Annex
- PD 6688-2: 2011

They contain the following requirements:

- Eurocode BS EN 1990 clause A2.4.3.2(2) requires comfort to be verified if the natural frequency is lower than 2.5 Hz for lateral and torsional modes;
- BS EN 1990 clause A2.4.3.2(1) states that comfort criteria should be defined in terms of maximum acceptable acceleration and proposes a horizontal limit for lateral and torsional vibrations of 0.2 ms^{-2} under normal use and 0.4 ms^{-2} for exceptional conditions, but makes these values nationally determined parameters;
- Clause NA.2.3.10 of the UK National Annex to BS EN 1990 states that the pedestrian comfort criteria should be as given in NA.2.44 of the UK National Annex to BS EN 1991-2. However, this clause does not specify a maximum acceptable acceleration for horizontal movement under normal use – it (and PD 6688-2) only address synchronous lateral vibration caused by lateral forces from footfall and does not address lateral and torsional modes excited by vertical loading.

None of the documents provide limiting horizontal accelerations for deliberate lateral shaking of the bridge.

A literal reading of all the applicable clauses therefore leads to the conclusion that a lateral-torsional mode with frequency less than 2.5 Hz should be verified for horizontal acceleration as BS EN 1990 clause A2.4.3.2 (2) still applies. However, no acceleration limit is provided as BS EN 1990 clause A2.4.3.2(1) is modified by the UK NA to BS EN 1991-2 which, itself, does not provide a limit.

Interim recommendations

Work is under way to update the relevant Eurocodes via BSI and CEN. However, the following interim recommendations are made until such time as the suite of codes above are made consistent.

- The design should conform to the requirements of BS EN 1990 clause A2.4.3.2(2) i.e. a verification of the comfort criteria should be performed if the fundamental frequency of the deck is less than 5 Hz for vertical vibrations, and 2.5 Hz for horizontal (lateral) and torsional vibrations.
- In the absence of a maximum acceptable acceleration for horizontal movement under normal use being specified by NA.2.44 of the UK National Annex to BS EN 1991-2, the recommended value given in BS EN 1990 clause A2.4.3.2(1) should be used (i.e. 0.2 ms^{-2}), measured at the level of the deck. The acceleration should be calculated under the vertical load models of NA.2.44 considering walking paths offset from the bridge centreline as necessary.
- Where the fundamental frequency of the bridge is less than 3 Hz for horizontal (lateral) and torsional vibrations, consideration should be given to making provision in the design, in discussion with the client, for possible installation of dampers to the bridge after its completion. (This recommendation makes some allowance for uncertainty in the value of damping and other parameters used in the calculations and also provides some potential remedy for unacceptable horizontal accelerations from deliberate shaking should they occur).
- Any further limiting criteria for pedestrian comfort, such as under deliberate shaking, should be determined on a project-by-project basis and agreed with the client.
- The potential for unstable lateral responses (synchronous lateral vibration) should still also be checked using NA.2.44.7 of the UK National Annex to BS EN 1991-2.

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AD 429

Slip factors for alkali-zinc silicate paint

This AD note draws attention to the slip factors for alkali-zinc silicate painted faying surfaces considered in AD 383 which have been updated in the 2018 revision of BS EN 1090-2.

AD 383, which was published in September 2014, discussed the slip factor for surfaces coated with alkali-zinc silicate paint and the significant influence of the coating thickness. The AD referred to forthcoming changes to Table 18 of BS EN 1090-2, expected to reflect concerns about the relationship between the coating thickness and slip factor. In the interim, AD 383 proposed slip factors of 0.3 (if certain recommended practices were followed) or 0.2 as a conservative value.

BS EN 1090-2 was revised in 2018 and slip factors are presented in Table 17. For surfaces coated with alkali-zinc silicate paint, the nominal thickness is now specified as $60 \mu\text{m}$, with a dry film thickness between $40 \mu\text{m}$ and $80 \mu\text{m}$.

If the applied coating meets the thickness limits specified in Table 17, a slip factor of 0.4 may be assumed. AD 383 noted that in practice the coating thickness can often exceed $80 \mu\text{m}$, so coating procedures will need to be carefully controlled and the dry film thickness measured, to ensure the limits in Table 17 are satisfied. If such control is not practical, then the conservative slip factors quoted in AD 383 may be adopted.

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AD 430

Wind load on unclad frames

The purpose of this note is to correct errors in BRE Special Digest SD5 which lead to the prediction of significantly higher wind loads on unclad frames than were calculated using the report which SD5 superseded.

BRE published Special Digest SD5 in July 2004. The document was produced principally because at the time, the current guidance for determining wind loads on frames, lattice structures and individual members was based on the BS code of practice CP3 Chapter V: Part 2 which had been withdrawn in October 2001. SD5 is based on BS 6399-2 and includes guidance on determining loads on individual members and lattice structures. It also includes a section on unclad building frames which is based on and intended to supersede BRE report BR173, Design guide for wind loads on unclad framed building structures during construction.

BR173 considers a series of identical parallel frames of overall width W at spacing S . The parameter S is used to select the appropriate normal force coefficient C_d according to the ratio W/S and the total solidity ratio denoted φ . In a given direction, φ is presented in BR173 as the sum of the horizontal and vertical solidity ratios: $\varphi = \varphi_v + \varphi_h$. In the direction perpendicular to the secondary beams, the horizontal solidity ratio used is the equivalent solidity ratio which allows for all the secondary beams in a bay denoted $\varphi = \varphi_v + \varphi_h^*$ (see item iii in the design example in BR173 para. 4.2.2). In SD5, the total solidity ratio is erroneously given as $\varphi = \varphi_v + \varphi_h + \varphi_{hs}$ i.e. the equivalent horizontal solidity ratio φ_{hs} is added to, instead of substituted for the horizontal solidity ratio φ_h . The total solidity ratio in this direction should be given in SD5 as $\varphi = \varphi_v + \varphi_{hs}$.

The spacing of the secondary beams is used in the determination of the equivalent solidity ratio for secondary beams. In BR173, this parameter is also denoted S and is likely to be different from the frame spacing but unfortunately, SD5 does not differentiate between the two parameters. In SD5, the relevant equation is no. 11: $\varphi_{hs} = (\varphi_1 + \varphi_2)\varphi_h$ where $\varphi_2 = (n - 1)(S/d - 7.5)/25$. According to BR173, in the expression for φ_2 the parameter S is the secondary beam spacing not the frame spacing.

The equivalent expression in BR173 is equation (4): $f_2 = (N_{bb} - 1)(S/b - 7.5)/25$. The secondary beam spacing should be used in the determination of φ_2 .

The approach to determining the wind load on unclad structures (lattice structures, frames and individual members) in SD5 (corrected as indicated) can also be used with BS EN 1991-1-4 and its UK National annex as the design pressures have identical target reliability to BS 6399-2.

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AD 431 Column web panel strengthening

The purpose of this Advisory Desk note is to draw attention to the contribution that full-depth stiffeners make to the shear resistance of column web panels.

SCI publication P398 covers the design of moment-resisting connections to Eurocode 3 and provides information on types of column strengthening in Table 2.1. Within this table, horizontal stiffeners are not credited with increasing the shear resistance of the web panel.

The special case of full depth stiffeners in both the tension zone and the compression zone is covered by clause 6.2.6.1(4) of BS EN 1993-1-8. This clause allows an additional contribution to the web panel shear resistance, based on the bending resistance of the flanges and the stiffeners which bound the web panel. The stiffeners and flanges can be envisaged as part of a Vierendeel truss, as shown in Figure 1.

If this additional contribution is to be utilised, the transverse stiffeners should be full depth and approximately the same width and thickness as the column flanges. The welds between the stiffeners and the flanges should be full strength, because the full plastic moment resistance of the stiffeners is assumed in the calculation.

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AD 432 Wind loads on building canopies

The purpose of this AD note is to direct designers' attention to PD 6688-1-4 as a source of design loads on building canopies and useful data and guidance relating to other topics.

A regular question for the SCI Advisory team relates to wind loading on canopies attached to buildings. A canopy may typically be provided over the entrance to a building, but questions arise as there are no coefficients provided in BS EN 1991-1-4.

Designers should refer to PD 6688-1-4, section 3.5, which provides force coefficients for canopies attached to the lower half of a building. Canopies attached to the upper half of a building should be assessed using the rules for free standing canopies fully blocked at one edge (the back or the side, depending on the wind direction). The forward reference in PD 6688-1-4 section 3.5 is incorrect – it should direct designers to section 7.3 of the Eurocode for loads on canopies.

It should be noted that when using the data provided in the PD, the reference height is the height of the building, not the height of the canopy. This is because gusts on the upper parts of the building can be directed down the building face onto the canopy.

The overall force coefficients tabulated in the PD in the downward direction are considerably larger than those in the Eurocode, particularly for shallow angle canopies attached at a relatively low level – so it is particularly important that the PD is consulted.

More generally, PD 6688-1-4 is a valuable resource with helpful guidance on such topics as non-simultaneous loads on faces, assessment of dominant openings, re-entrant corners and inset faces.

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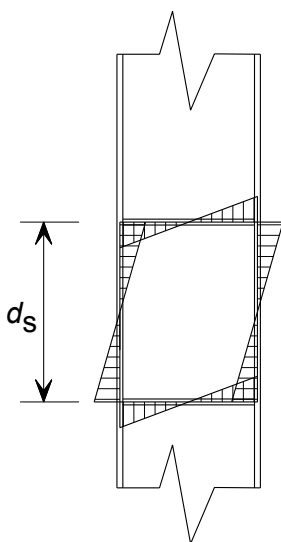


Figure 1: Vierendeel bending around column web panel

AD 433 Dynamic modulus of concrete for floor vibration analysis

The purpose of this AD note is to provide advice on the choice of elastic modulus of concrete when undertaking the vibration analysis of a composite floor.

The elastic modulus of concrete depends on the constituent materials of the concrete mix and on the age of the concrete. It also depends on the duration of loading and whether the concrete is assumed to be cracked or un-cracked. Table 3.1 in BS EN 1992-1-1 gives strength and deformation characteristics for concrete by strength class. The values are tabulated for normal weight concrete with quartzite aggregates and are based on the cylinder strength f_{ck} at 28 days. The formula for the secant modulus E_{cm} is: $E_{cm} = 22[(f_{ck} + 8)/10]^{0.3}$.

The value is in GPa when the cylinder strength is in MPa. Adjustments to the values for quartzite aggregates are given for limestone, sandstone and basalt aggregates. Practice in continental Europe is to use a dynamic modulus based on E_{cm} enhanced by 10%¹.

In UK practice, values for elastic modulus determined from the code are not considered suitable for the calculation of beam deflections from which the natural frequency of the beam is to be determined. The dynamic behaviour generally involves small amplitude vibrations to which the secant modulus at 28 days E_{cm} is not relevant. Instead, given the uncertainty regarding the parameters which affect the actual properties of concrete (type of aggregate, age of concrete, compressive strength etc.), an approximate dynamic modulus should be used which (from practice) gives reasonable results.

SCI publication P354 *Design of floors for vibration: a new approach*² and Concrete Centre publication: *A design guide for footfall induced vibration of structures*³, both recommend the same values for the dynamic modulus of concrete which is appropriate for the estimation of the dynamic response of composite or concrete structures. Values are given for normal weight and light weight concrete as follows:

Uncracked concrete	Dynamic modulus (GPa)
Light weight	22.0
Normal weight	38.0

When using references 2 and 3, the stated values for dynamic modulus should not be enhanced by 10%.

References

1. European Commission – Technical Steel Research: *Generalisation of criteria for floor vibrations for industrial, office, residential and public building and gymnastic halls*, RFCs; Report EUR 21972 EN, ISBN 92-79-01705-5, 2006.

2. Smith, A L, Hicks, S J, Devine P J, *Design of floors for vibration: a new approach*, Revised edition, February 2009, SCI publication P354
3. Willford, M R, Young, P, *A design guide for footfall induced vibration of structures*, Concrete Centre, November 2006

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AD 434

Validity rules for hollow section joints

This AD note concerns a significant typographical error in Table 7.8 of BS EN 1993-1-8. The table presents validity limits for welded joints between hollow section brace members and RHS chord members.

For the common case of a K or N overlap, there is a limitation under the "Gap or overlap" column expressed as:

$$b_i/b_j \leq 0.75$$

where b_i is the width of the overlapping bracing member, and b_j is the width of the overlapped member. Thus the limit precludes bracing members of the same size, for which $b_i/b_j = 1$, and this is clearly wrong.

In fact, the limit should be expressed as:

$$b_i/b_j \geq 0.75$$

which prevents a narrow overlapping bracing being welded to a wide overlapped brace, but permits bracing of the same width to be used.

This limit is correctly expressed in literature published by Tata Steel, and has been corrected in the draft revisions to EN 1993-1-8.

As an aside, it may assist designers to note that definitions of some factors that appear in the joint verification expressions, such as β , λ_{ov} , n and γ are found in Section 1.5 of BS EN 1993-1-8, not in Section 7 as might be expected. Similarly the definition and dimensions of gap and overlap joints are found in Figure 1.3, rather than in the section concerned with hollow section joints.

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AD 435

Beams supporting precast planks: checks in the temporary condition

The purpose of this note is to remind designers of their responsibility for basing their design on a safe method of erection. This is particularly necessary if structural stability in the part-erected condition is not evident.

The CDM (2015) regulations consider this in Regulation 11 where "(1) The principal designer must ... ensure that, so far as is reasonably practicable, the project is carried out without risks to health or safety. ..."

"In fulfilling the duties in paragraph (1), the principal designer must identify and eliminate or control, so far as is reasonably practicable, foreseeable risks to the health or safety of any person –

(a) carrying out or liable to be affected by construction work; ..."

BS EN 1090-2:2018 addresses this issue more directly in paragraph 9.3.1 which states that the design basis method of erection shall consider amongst other things the following: "d) stability concept for the part-erected structure including any requirements for temporary bracing or propping".

SCI publication P401: Design of composite beams using precast concrete slabs in accordance with Eurocode 4 states in Section 3.6 "The stability of the steel beams during the erection of the floor units and the placement of the structural topping must be considered. The designer should take due account of the floor erection process (which will usually require erection in 'bays' to avoid excessive re-siting of the crane). Should a particular sequence of erection or temporary support be necessary, this should be noted in the specification and on the drawings. The placement of the precast concrete units should be carefully controlled in order that out of balance construction loads are kept within the limits assumed in the beam design ..."

Section 4 of the publication discusses the checks for torsion which should be carried out in the event that an out-of-balance load results from the assumed erection sequence. Such conditions may result from:

1. The assumed erection sequence;
2. Unequal plank spans on either side of the beam;
3. Planks spanning in different directions on either side of the beam;
4. The sequence of placing the in-situ topping.

Other relevant issues are the effectiveness of the lateral restraint provided by the precast planks and the specification of additional restraint if the planks are inadequate by themselves. (See P401, Section 3.6).

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The screenshot shows the website interface with a search bar at the top. Below the search bar, there is a navigation menu with options like Home, Projects and Features, News, Technical, SSSA, 50 & 20 Years Ago, and DigiMags. A prominent red button says "SUBSCRIBE to NSC CLICK HERE". The main content area features a large image of a steel structure under construction with the headline "Historic hangar takes off, again". To the left, there is a "News" section with several article thumbnails and titles, such as "Sub-contractors join forces for safe ground conditions" and "Steel completes on Birmingham Conservatoire". On the right, there is an "NSC ARCHIVE" section and a "THIS MONTH'S MAGAZINE" section with a "CLICK HERE" button. At the bottom, there is a "THIS MONTH'S FEATURES" section with three small images and titles: "Steel provides energy solution", "Town centre regenerates with steel", and "Historic hangar takes off, again".

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