Using Steel Grating Platform as Stabilization for Steel Primary I-Beam

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Subject Using Steel Grating Platform as Stabilization for Steel Primary I-Beam

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ABSTRACT

One of the basic steps of steel beam design is to define the effective lateral-torsional buckling length of structural elements. According to different kinds of supports of the beam, the effective buckling length can be calculated sketchily by using Euler's formula. However, the stiffness of steel gratings together with secondary beams can resist the lateral-torsional buckling of platform primary beams and affect their effective buckling lengths.

The purpose of this Bachelor's thesis was to build and analyse various structural models of typical steel platforms to determine the effective buckling lengths of platform narrow flange primary I-beams. The results show that gratings and secondary beams provide significant lateral restraint to narrow flange primary I-beams and reduce their effective buckling lengths. But the restraining effects from grating is limited by the properties of beams, stiffness of beam to beam connection and the size of the grating platform. Some additional tests of wider flange primary I-beams were also calculated as a reference to solve how effective stabilization grating platform can provide for wider flange primary beams.

Keywords effective buckling length, steel grating, lateral-torsional buckling, steel I-beam, steel design

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1 INTRODUCTION

1.1 Background

When an unrestrained I-beam is in flexure, the compression top flange tends to move laterally, but the web and tension bottom flange will provide restraints to prevent this behavior. However, when the flexure load arrives to a certain limit, the compression top flange will buckle locally and make the I-beam suffer from lateral-torsional buckling.

Steel gratings are used widely in industrial buildings for walkways or platforms, but when engineers are designing the supporting beams for steel gratings, the interaction between steel gratings and supporting beams is difficult to define and normally this interaction is ignored. However, if steel gratings can provide adequate restraints to prevent the supporting beams from lateral-torsional buckling and reduce their effective buckling lengths, then the profile of beams can be changed to smaller ones to save costs and spaces required for platform structures. Figure 1 below shows a steel grating platform.

Figure 1. Steel grating platform (Access/Stainless Steel Grating - Flooring, Platforms, Access Systems, n.d.)

1.2 Idea of the research

There are already good studies about how steel gratings affect the lateral-torsional buckling of secondary beams directly connected to gratings, for example, Stabilisierung von I-Trägern durch Gitterroste (Gilde, 2003), so this research will mainly focus on the supporting primary beams.

In steel design, the stiffness of beam is very important. When comparing narrow flange I-beams (the ratio of height and width of the beam is more than 1.8) with wider flange I-beams, having the same stiffness, the weight of the first one is much smaller than the second one, which means that narrow flange beams are more economic. But the narrow flange beam has lower capacity due to lateral-torsional buckling in case there is no horizontal / torsional support at the top flange of the beam. If the gratings together with secondary beams can provide adequate resistance to the lateral-torsional buckling of narrow flange primary I-beams, then they can be considered in designing more economical grating platforms.

The effects from steel grating floor (gratings + secondary beams supporting the gratings) to the lateral-torsional buckling of beams can be indicated with the effective buckling lengths of beams. It is not possible to calculate the critical lateral-torsional buckling moment of a beam by using simple analytical formulas for this kind of complex structure. Finite Element Analysis (FEA) is suitable for this structure, but this time-consuming method is not possible to be used in daily design work. Therefore, the accurate FEA models are built to define the approachable value of the effective buckling length which can be used to define the critical buckling moment of a beam by using simple analytical formulas.

According to NCCI(SN003a-EN-EU), the relationship between the elastic critical buckling moment and effective buckling length for a doubly symmetric cross-section beam can be shown as formula:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 GI_t}{\pi^2 EI_z} + \left(C_2 z_g\right)^2} - C_2 z_g \right\} \qquad (1)$$

In this formula, kL is the effective buckling length and k is the effective length factor. But in this research, the effective buckling length is represented by L_{cr} and the effective buckling factor is represented by k_{LT} to avoid confusion.

According to formula (1), it is possible to define the effective buckling length of a beam by calculating the elastic critical buckling moment.
Therefore, the effective buckling length of the primary beam of analyzed typical platforms calculation is separated into two parts:

1. Calculation of the elastic critical buckling moment (M_{cr}) by FEA (Finite Element Analysis)
2. Calculation of the effective buckling length (L_{cr}) and the effective length factor (k_{LT}) of the beam based on NCCI (NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU).

The target is to find how efficiently the steel gratings affect the effective buckling length of narrow flange primary I-beams in different sizes of grating platforms. The size of the grating platform and the beam sizes for the design loads should be designed based on real design rules; therefore, it is not possible to calculate all the possible platform configurations. To get a good overall understanding of the problem, eight typical sizes of platforms have been chosen from real projects to provide practical examples. The profiles of the secondary and primary beams are designed according to the geometry and the loading of the platform.

For references, there are also extra tests done to see of how the effective length factor will change if the flange of the primary beam is wider. All the results are collected and analyzed to give practical and safe suggestions for the effective length factor for grating platform primary I-beams in steel design.

2 THE BASIS OF THE RESEARCH

2.1 Connections between secondary beams and primary beams

There are normally three different kinds of connections between the secondary and primary beams:

1. When the secondary beam is on the top of the primary beam (connecting with two bolts in bidiagonal direction) (Figure 2)

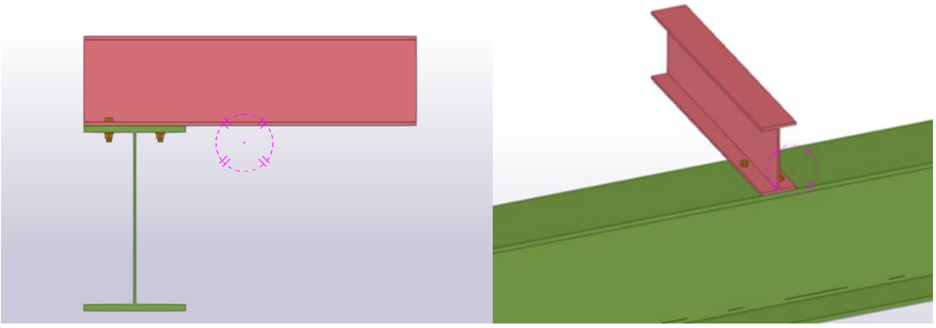


Figure 2. Condition 1 of the connection between the secondary beam and the primary beam (side view and 3D view)

2. When the top levels of the secondary and primary beams are same (connecting with two bolts and one stiffener plate) (Figure 3)

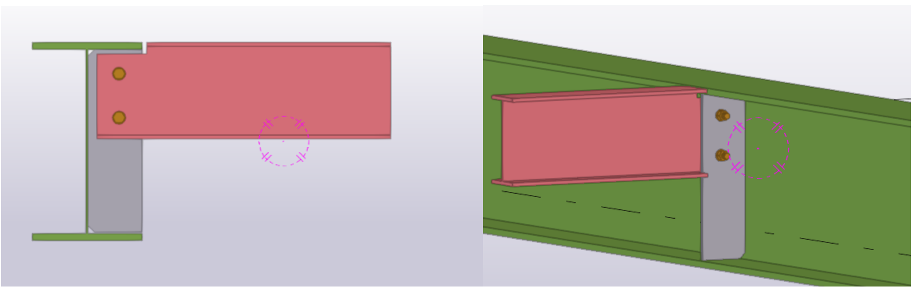


Figure 3. Condition 2 of the connection between the secondary beam and the primary beam (side view and 3D view)

3. When the top levels of the secondary and primary beams are same (connecting with 4 bolts and one end plate in the secondary beam) (Figure 4)

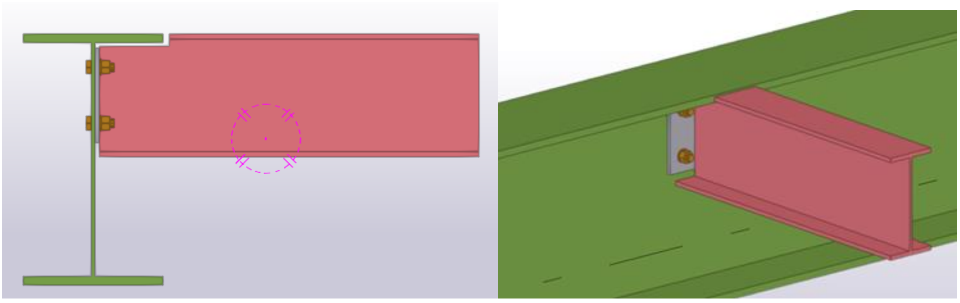


Figure 4. Condition 3 of the connection between the secondary beam and the primary beam (side view and 3D view)

The research will be based on the condition 1 (when the secondary beam is on the top of primary beam). As for condition 2, the stiffener plate can provide quite good stiffness in connection to the primary beams to avoid lateral-torsional buckling. We can assume that there is at least the same stability in the structures in condition 2 as in condition 1. Thus, the results from condition 1 can be applied safely on the condition 2. And as for condition 3, there will be extra stiffness calculations for connection between secondary beam and primary beam to evaluate if the results of research can also be applied to case 3.

2.2 Design Criteria of Loads, beams and grating panels

2.2.1 Loads

Permanent and live loads are considered to the following:

- permanent load g=0.3 kN/m^2 (weight of steel grating)
- live load q=4.0 kN/m^2

2.2.2 Beams and grating panels

The following lists the technical details of beams and grating panels:

- The ratio of height and width of primary beam is 1.8 or more than 1.8. (h/b≥1.8)
- Secondary beams are on the top of primary beams and connected to the top flanges of primary beams with two bolts in bidiagonal direction.
- Top flanges of all the secondary beam are at the same level. Spacing of secondary beams is 1.2m, which is suitable for typical grating panels for loading of 4.0 kN/m^2.
- Grating panels shall be continuous over at least 3 secondary beams and every grating panel is connected to each beam with bolts (M8). The self-tapping screw in the connection shall go through top flange of the beam (see Figure 5 and Figure 11).

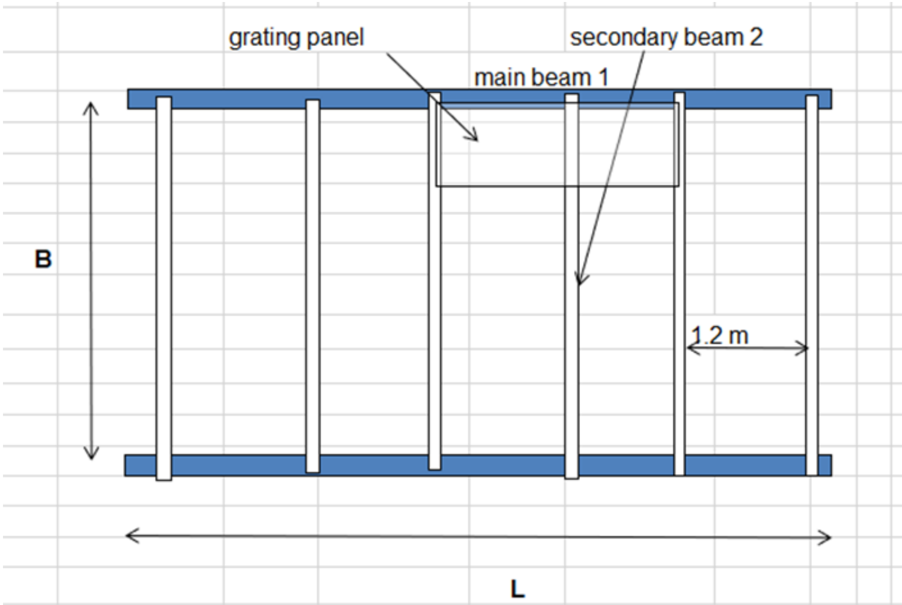


Figure 5. Plan view of simple model of steel grating and beams

- The width of grating panel is 1m, and the length of grating panel is 2.4m. The profile of steel grating: bearing bar 30x3 c/c33, 6x6 c/c75. Size of grating panel can be different on one the side of the model according to the size of platform. But the bearing direction of grating panel is always parallel to primary beams in the RFEM model.
- Deflection limits of beams are shown in Table 1:

Table 1. Deflection limits of beams

	For total load	For live load
span of primary beam up to 6m	L/300	L/350
span of primary beam up to 8.4m and over	L/350	L/400
secondary beam	L/250	-

- steel grades of beams are shown in Table 2:

Table 2. Steel grades of beams

secondary beam	f_y=235 N/mm^2
primary beam	f_y=355 N/mm^2

- Steel shapes: European rolled I-profile or welded I-profile.
- Sizes of secondary beams are calculated assuming one lateral-torsional support at mid span of the beam. (provided by grating)
- Primary beams are designed based on design assumption: 1. one lateral support at mid span of the beam. 2. secondary beams together with gratings provide this lateral restraint.

An Excel sheet provided by the company Sweco for calculating the sizes of the profiles of beams was used. It is assumed that gratings can reduce the effective lateral-torsional buckling length factor of the beam to around 0.5, so in the predesign of the profiles of secondary and primary beams, the effective length factors are assumed to be 0.5. Any other calculations are based on Eurocode. For examples of Excel sheet calculations, please check Appendix 1 and 2.

(Design criteria of loads and beams are made by Risto Nurminen from Sweco)

2.2.3 Beam profiles for different platform configuration

Beam profiles are shown in Table 3. Beam 1 is the primary beam, and Beam 2 is the secondary beam.

Table 3. Beam profiles for different platform configuration

Span	B = width of platform (m)					
L (m)	2,0		4,2		6,2	
	Beam 1	Beam 2	Beam 1	Beam 2	Beam 1	Beam 2
6,0	IPE200	IPE140	IPE240	IPE160		
8,4	IPE270	IPE140	IPE360	IPE160	WI400-5-12*220	IPE220
10,8	IPE360	IPE140	WI400-5-15*220	IPE160	WI450-5-16*250	IPE220

The lengths in Table 3 are the spans of platforms. In RFEM models, all the secondary beams are stretched to the edge of primary beams, and all the primary beams are stretched to the edge of secondary beams. Therefore, the real lengths of beams in RFEM models are slightly longer than the span according to their profiles. Stretching of beam is necessary to be able describe behaviour of the beams and connection stiffness correctly in analysis model.

2.3 Bolt size and bolt distance

The size of bolt and bolt distance of connections between primary beams and secondary beams are different due to different profiles of beams as shown in Figure 6 and Tables 4,5 and 6 below.

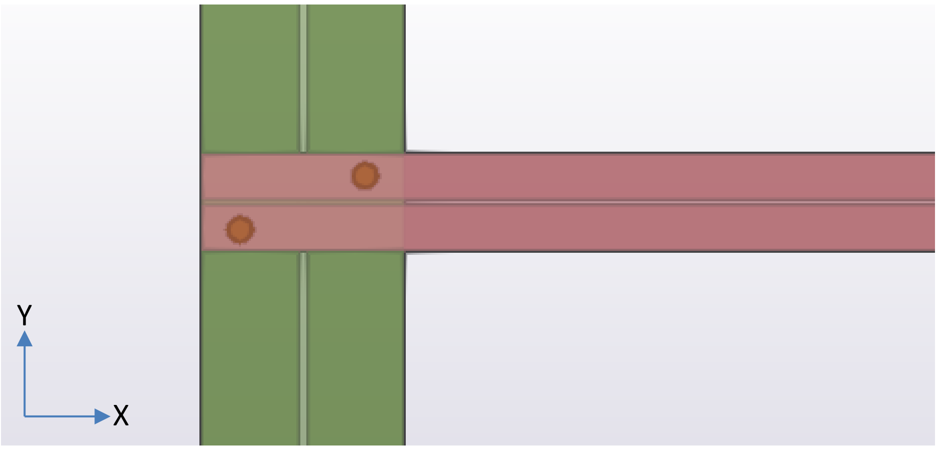


Figure 6. Bolts location of secondary beam (IPE160) for Beam 1-IPE360 /Beam 2-IPE160 (example)

Table 4. Bolt sizes and distances of connections between primary beams and secondary beams (Beam 2-IPE140)

	Bolt size	Bolt distance (x-y)
IPE220	M10	60mm-40mm
IPE270	M10	85mm-40mm
IPE360	M10	105mm-40mm

Table 5. Bolt sizes and distances of connections between primary beams and secondary beams (Beam 2-IPE160)

	Bolt size	Bolt distance (x-y)
IPE240	M12	75mm-45mm
IPE360	M12	105mm-45mm
WI400-5-15*220	M12	110mm-45mm

Table 6. Bolt sizes and distances of connections between primary beams and secondary beams (Beam 2-IPE220)

	Bolt size	Bolt distance (x-y)
WI400-5-12*220	M12	110mm-60mm
WI450-5-16*250	M12	125mm-60mm

3 CALCULATION OF THE ELASTIC CRITICAL BUCKLING MOMENT BY FEA

3.1 Background

One of the most important and hardest parts of this research is to simulate the situations as realistic as possible. There are many important elements that have to be considered during calculation: the stiffness of one grating panel, connections between grating panels and secondary beams and connections between beams. It is not possible to calculate everything using simple analytical formulas. However, Finite Element Analysis (FEA) is suitable for complex structure calculations.

Finite Element Analysis is a numerical method to solve boundary value problems. It is mainly used for structural analysis, heat transfer, flow calculation and acoustics. In this research, only structural analysis is used. It can be done by building FEM models consisting of trusses, beams, shell elements or solid elements in RFEM programme.

Stability analysis in RFEM makes it possible to calculate the elastic critical load factor of a beam. In this analysis, the model is considered as a linear analysis and the material model is in elastic domain. However, the imperfections are not considered, and the results are not related to resistance in this analysis. The resistance of the parts can be defined later. For example, after the effective buckling length of the beam is defined, the initial imperfections are considered when calculating the resistance according to Eurocode. Some elements need be simplified due to calculation time: 1. connections between secondary and primary beams are modelled by using rigid elements with adequate spring stiffness to simulate real connection stiffness. 2. grating panels are modelled by using orthotropic material.

3.2 The geometry of RFEM model

The examples of the geometry of RFEM model are shown in Figures 7 and 8. In the following there is some general information of the geometry of RFEM model:

- There are two primary beams.
- The number of secondary beams is decided by the span of grating platform.
- Grating panels are modelled by using orthotropic material.
- In order to connect all the parts together, beams are modelled using shell elements. The thicknesses of shell elements in webs and flanges correspond to the dimensions of beam profiles. The shell planes of beam are located at the middle of surfaces of webs and flanges.
- The connections between grating panels and secondary beams are modelled using beam elements to simulate the stiffness of the bolts.
- There is 10 mm gap between steel grating panels to make the panels work separately. Because of this gap, the surface of the top flange of the secondary beam is divided again to connect with the bolts.
- The connections between secondary and primary beams are modelled using rigid elements with adequate spring stiffness to simulate stiffness of connections.
- All the cases are modelled separately.

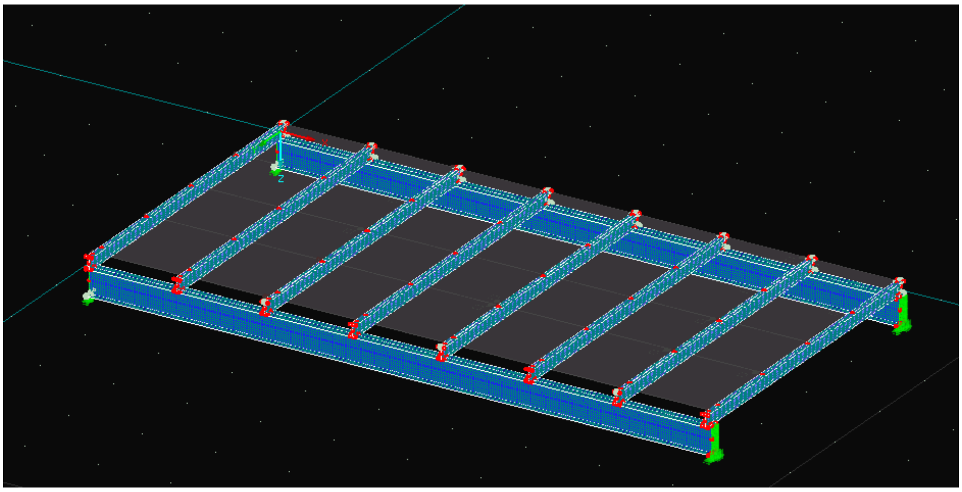


Figure 7. Overview of RFEM model Beam 1-IPE360 /Beam 2-IPE160 without grating panels (example)

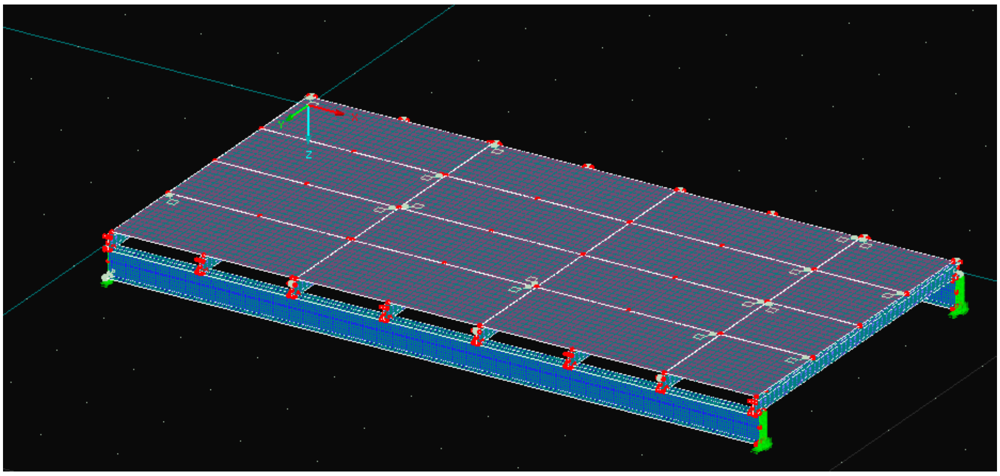


Figure 8. Overview of RFEM model Beam 1-IPE360 /Beam 2-IPE160 with grating panels (example)

3.3 The Material model of a steel grating panel

For grating type of structure, the stiffness of the grating panels is different if the load is applied in different directions. It is not possible to build accurate model using solid elements to describe the real stiffness behaviour of grating due to the limited time. The commercial software usually does not offer a direct method of modelling the stiffness behaviour of grating with shell elements. To simplify the process of modelling and optimize the accuracy of results, the steel grating panel was modelled by using orthotropic material model and the stiffness of grating was calculated separately by Henri Hautamäki (Sweco) using Ansys programme. The profile of steel grating: bearing bar 30x3 c/C33, 6x6 c/c75.The material properties of plate are calibrated to equal grating stiffness. This is a typical profile of steel grating panel which is generally used in industry buildings in the company. The Ansys model is solid model with real geometry of grating. Then the grating is loaded by three different load cases (see Figure 9) and the strains was solved.

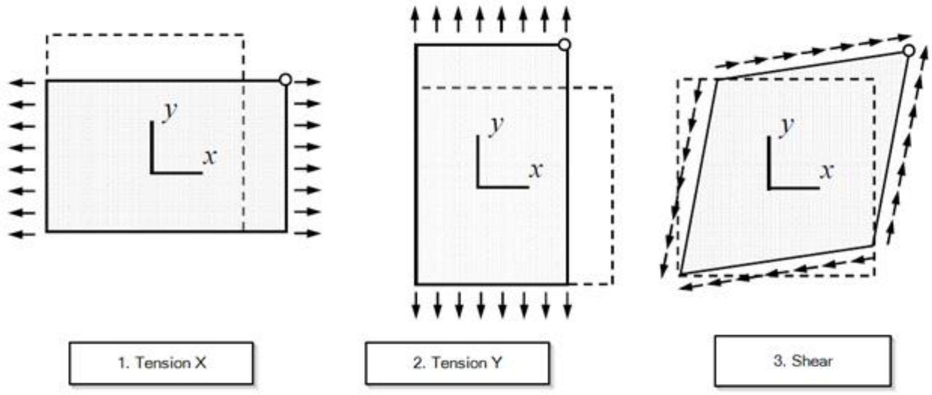


Figure 9. Load cases of steel grating

The plate thickness of 30mm is used to calculate the equivalent E, G and v_{xy}. Basic equations for 2d orthotropic material are shown in the equations:

$$\begin{Bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{Bmatrix} = \begin{bmatrix} \frac{1}{E_1} & -\frac{\nu_{12}}{E_1} & 0 \\ -\frac{\nu_{21}}{E_2} & \frac{1}{E_2} & 0 \\ 0 & 0 & \frac{1}{G_{12}} \end{bmatrix} \begin{Bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_{12} \end{Bmatrix} \qquad (2)$$

$$\frac{\nu_{12}}{E_1} = \frac{\nu_{21}}{E_2} \qquad (3)$$

The unit force of 1000N was used to calculate strains (ε_x, ε_y, ε_{y2}, ε_{x2}) by using real geometry model. (for detailing calculation, please check Appendix 3)
The results of Appendix 3 are applied in RFEM model (see Figure 10) to define the stiffness of steel grating panels.

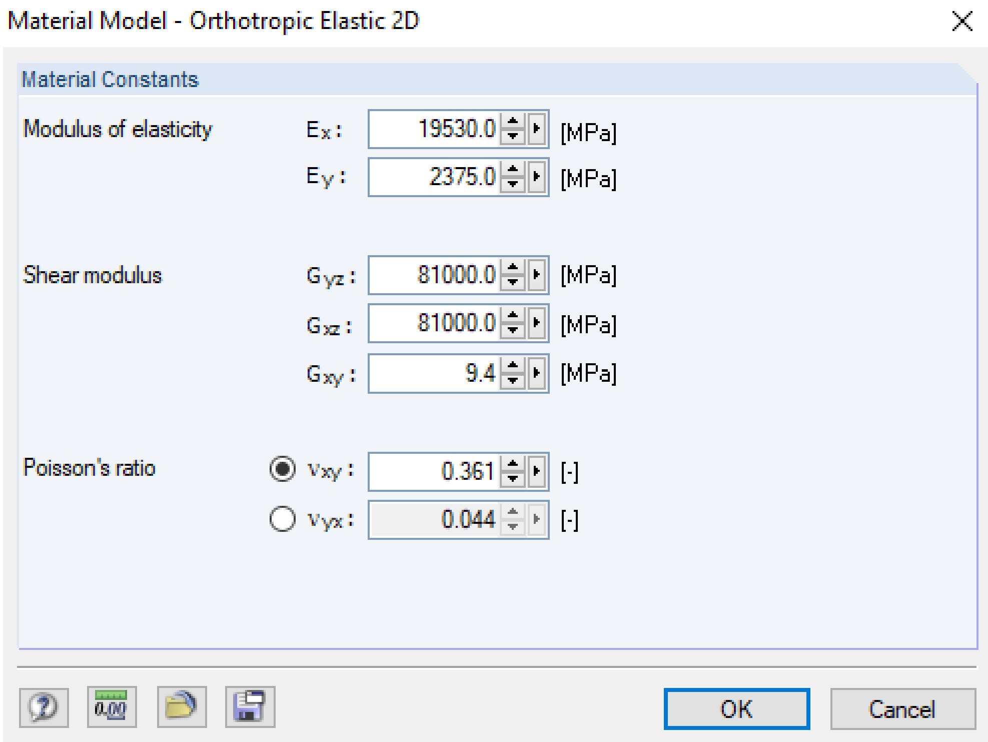


Figure 10. Material constants of steel grating in RFEM model

The grating panels are modelled by using Orthotropic elastic 2d material model in RFEM, and the orthotropy type is constant thickness of 30mm. The weight of the grating panel surface is applied as 0 kg/m^3 in the setting and the actual weight of the grating will be applied as permanent load on the secondary beams. Global x direction is the bearing direction of steel grating in RFEM model.

3.4 Connections between steel gratings and secondary beams

The connections between steel gratings and secondary beams are modelled using beam elements. These beams act like a cantilever. They can transfer load to the top of grating via the saddle as shown in Figure 11. Therefore, in the model, the top end of the bolt is pinned, and the bottom end of the bolt is rigid (see Figure 12).

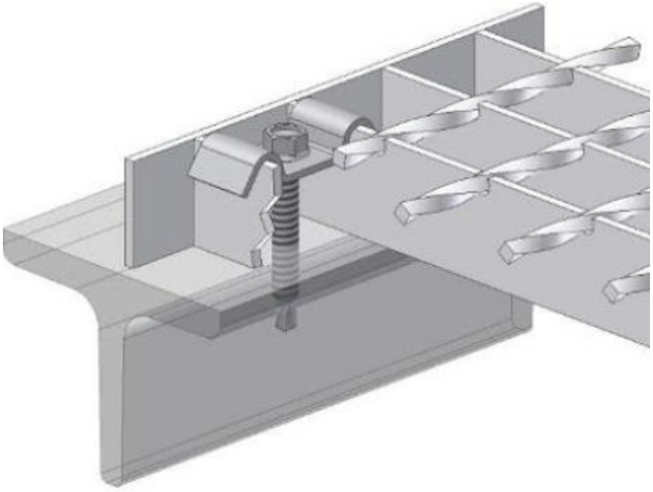


Figure 11. Connection with saddle clip between gratings and beams in real situation (Fixings, n.d.)

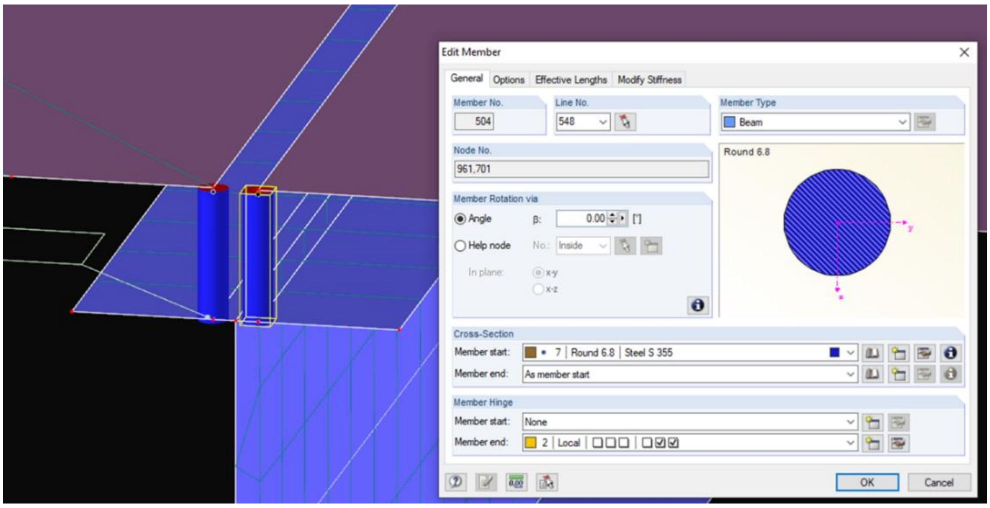


Figure 12. Properties of the bolt between steel grating and secondary beam RFEM model Beam 1-IPE360 /Beam 2-IPE160 (example)

The length of the bolt is 30mm as same as the thickness of the steel grating. M8 bolts are used in this situation. But in real situation, the bolt is with full length threads, so the cross-section properties of the beam element correspond to the nominal stress area properties. The diameter can be calculated from the stress area. For M8 bolt, the effective area is 36.6mm^2 as shown in Table 7; therefore, the diameter of the bolt in RFEM model is 6.8mm.

Table 7. Part of Minimum ultimate tensile loads — ISO metric coarse pitch thread (International standard ISO 898-1, 2009-04-01)

Thread [a] d	Nominal stress area $A_{s,nom}$ [b] mm²	Property class 4.6	4.8	5.6	5.8	6.8	8.8
		Minimum ultimate tensile load, $F_{m\,min}$ ($A_{s,\,nom}$ ×					
M3	5,03	2 010	2 110	2 510	2 620	3 020	4 020
M3,5	6,78	2 710	2 850	3 390	3 530	4 070	5 420
M4	8,78	3 510	3 690	4 390	4 570	5 270	7 020
M5	14,2	5 680	5 960	7 100	7 380	8 520	11 350
M6	20,1	8 040	8 440	10 000	10 400	12 100	16 100
M7	28,9	11 600	12 100	14 400	15 000	17 300	23 100
M8	36,6	14 600 [c]	15 400	18 300 [c]	19 000	22 000	29 200 [c]
M10	58	23 200 [c]	24 400	29 000 [c]	30 200	34 800	46 400 [c]
M12	84,3	33 700	35 400	42 200	43 800	50 600	67 400 [d]

3.5 Connections between secondary and primary beams

In reality, the connection between primary and secondary is made with a bolt through flanges. This connection is not fully rigid, so the rotation stiffness of connection needs to be considered. In RFEM model, the bolt and the contact between flanges are not modelled according to reality. The connection is simplified using five rigid elements (see Figure 13). The stiffness of the whole connection is applied in one point, so the rigid elements at the plane of flange are used to extend stiffness to the whole connection area. This makes the stiffness distribution of connection more reliable. The rotation stiffness of connection is calculated separately and applied in one of the rigid elements as spring stiffness.

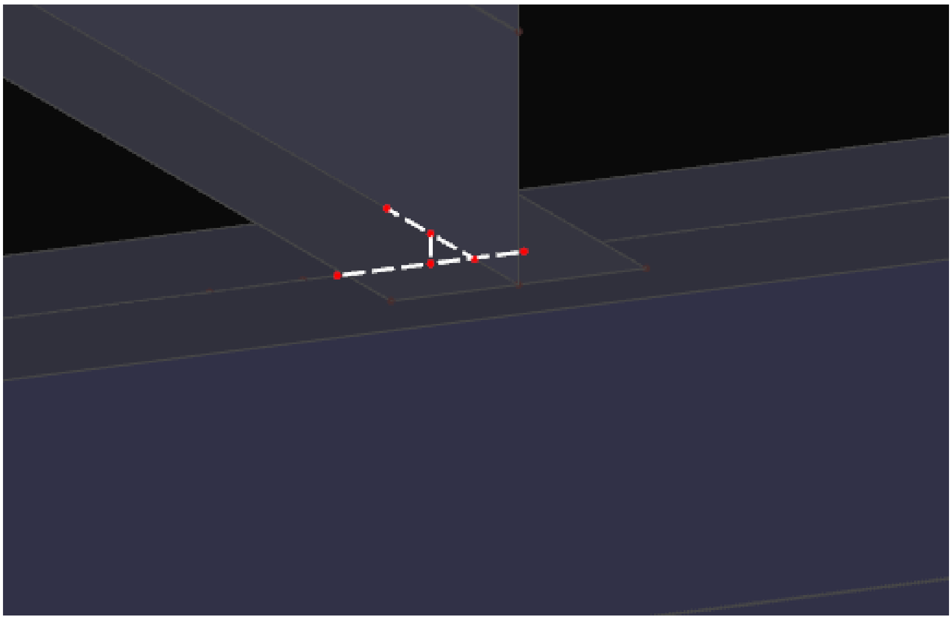


Figure 13. Five elements between secondary and primary beams in RFEM model Beam 1-IPE360 /Beam 2-IPE160 (example)

The maximum length of rigid element at flange level should be no longer than the width of the flange of the adjoining beam profile. In all the RFEM models. all the rigid elements on flanges are 30mm long for saving the time of modelling. Both end points of these four rigid elements in flange planes are rigid.

The short rigid element in the middle should be 0.5*thickness of the flange of the primary beam + 0.5*thickness of the flange of the secondary beam long. The bottom point of the element which connect to the flange of primary beam is rigid. The spring stiffness of the top point defines the initial stiffness of the connection (see example in Figure 14). The initial stiffness is calculated by using commercial programme IDEA StatiCa (see examples in Figures 15 and 16 and Tables 8 and 9).

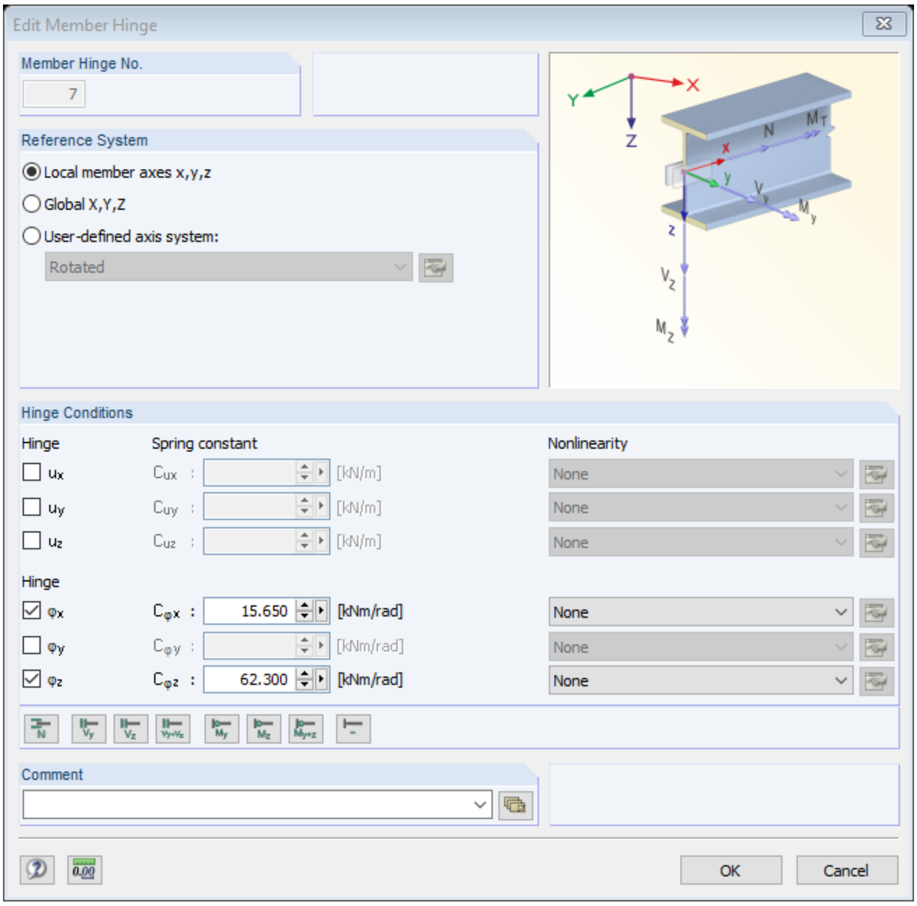


Figure 14. Stiffness of the top point of the short middle rigid element in RFEM model Beam 1-IPE360 /Beam 2-IPE160 (example)

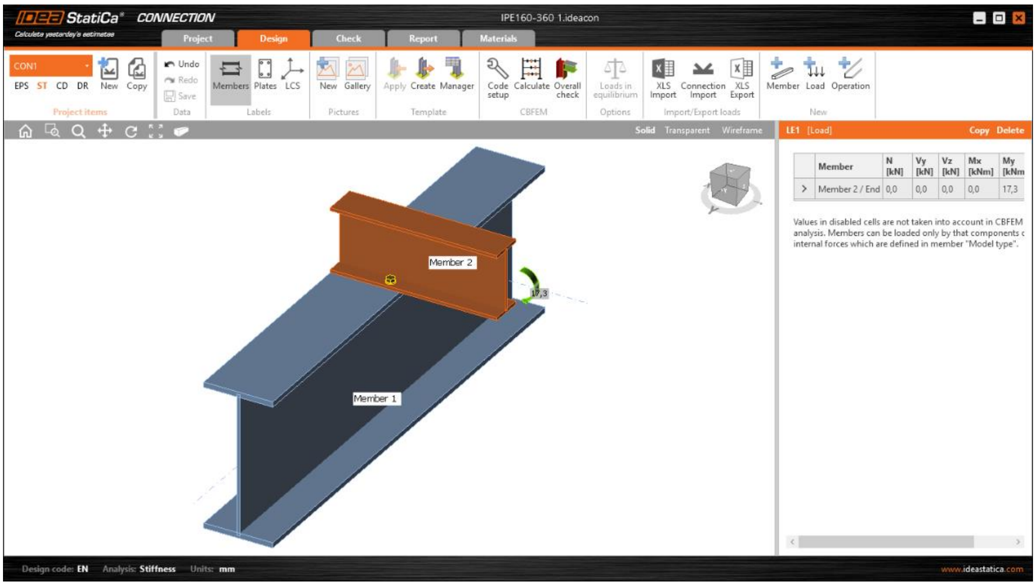


Figure 15. Stiffness calculation model in IDEA StatiCa of Beam 1-IPE360 /Beam 2-IPE160 in local z direction (example)

Table 8. Part of the report of stiffness calculation model in IDEA StatiCa of Beam 1-IPE360 /Beam 2-IPE160 in local z direction (example)

Rotational stiffness

Name	Comp.	Loads	Mj,Rd [kNm]	Sj,ini [kNm/rad]	Φc [mrad]	L [m]	Sj,R [kNm/rad]	Sj,P [kNm/rad]	Class.
Member 2	My	LE1	5,0	62,3	476,5	4,30	10615,1	212,3	Pinned

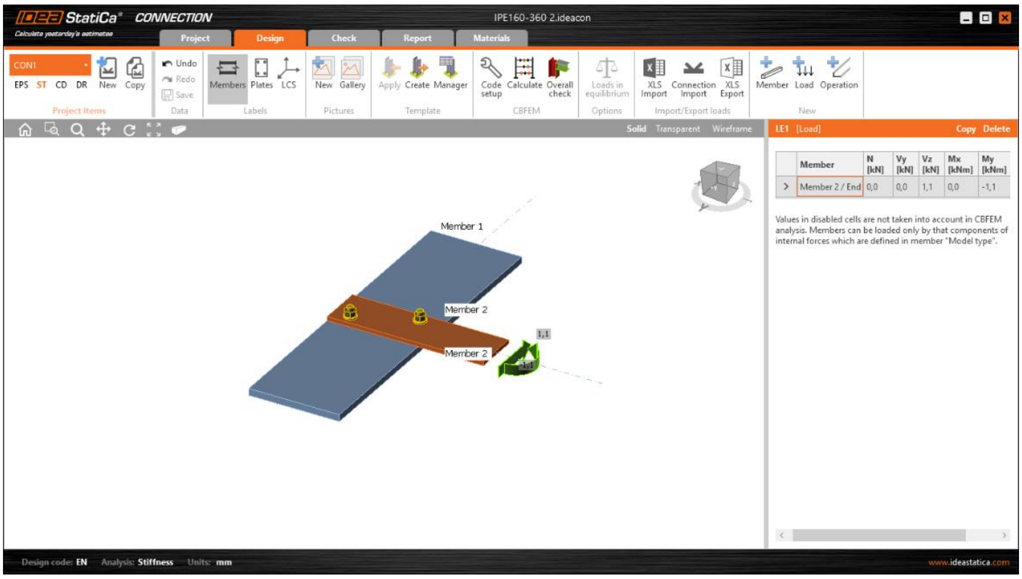


Figure 16. Stiffness calculation model in IDEA StatiCa of Beam 1-IPE360 /Beam 2-IPE160 in local x direction (example)

Table 9. Part of the report of stiffness calculation model in IDEA StatiCa of Beam 1-IPE360 /Beam 2-IPE160 in local x direction (example)

Rotational stiffness

Name	Comp.	Loads	Mj,Rd [kNm]	Sj,ini [kNm/rad]	Φc [mrad]	L [m]	Sj,R [kNm/rad]	Sj,P [kNm/rad]	Class.
Member 2	My	LE1	-3,6	∞	-230,0	4,30	3,4	0,1	Rigid

It is not possible for IDEA StatiCa to calculate the initial stiffness value around x-axis, but it is possible to define this value by ϕc/Mj.Rd expression. Actually, there is error in calculating the stiffness in local x direction, because it is not possible to define the shear force or moment in this direction. But according to tests, the margin of this error on the effective buckling length of the beam is less than 1%, and this error has no or minimal effect on the effective buckling factor.

After applying the results from IDEA StatiCa to RFEM model, the secondary beam can only move around this one point with correct stiffnesses. All the stiffnesses of this point in all the models are calculated by this process one by one to make the model more accurate.

3.6 Supports of the primary beam

The primary beam is supported by pinned support. To be more precisely, it is supported by fork supports. Fork supports allow warping to develop freely, but transverse displacement is prevented. Usually, the standards and general instructions are based on this type of support. This correspond to the behaviour of end plate connection. There are three different kinds of supports used for the primary beam:

1. Line support on the web---simple support in global y direction.
2. Line support on the bottom flange---simple support in global z and y direction.
3. Node support in the middle of bottom flange---simple support in global x direction.

The example of the supports is shown in Figure 17.

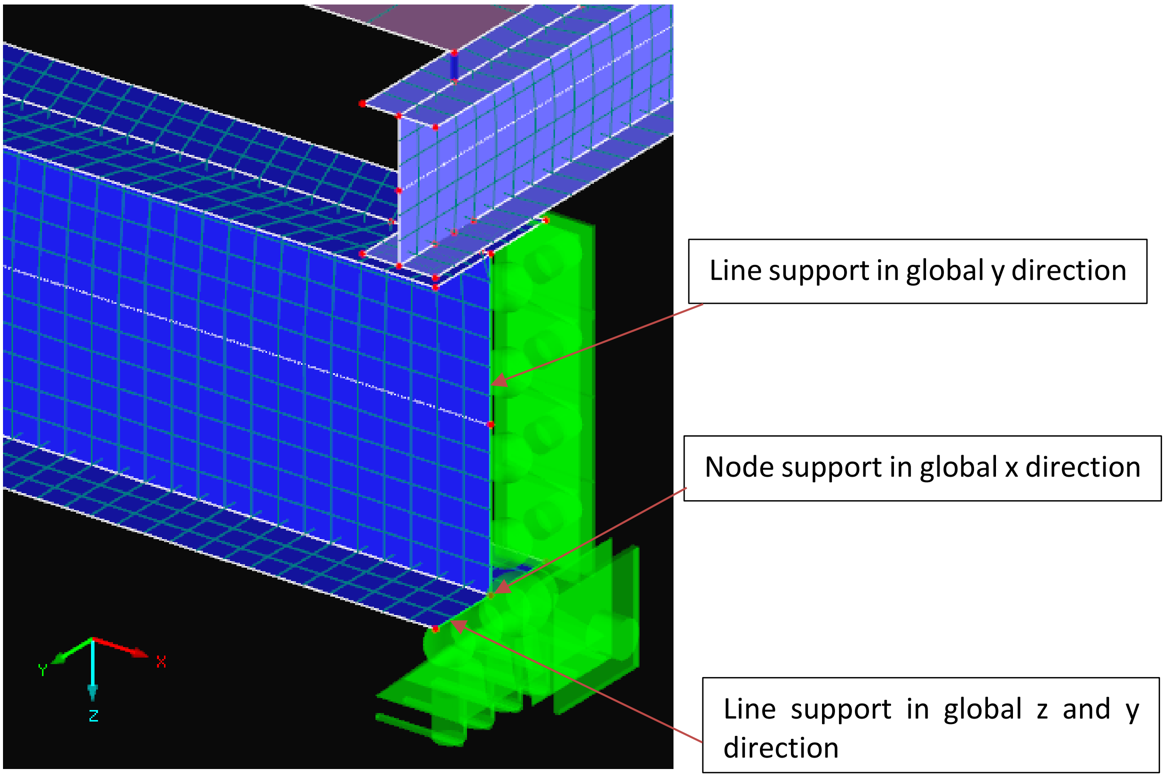


Figure 17. "Fork supports" of primary beam in RFEM model Beam 1-IPE360 /Beam 2-IPE160 (example)

The x-direction support is located only on one end of the primary beam to allow the beam to expand freely.

3.7 Stability Analysis in RFEM model

After all the details are done correctly in the model, stability analysis can be used to calculate the critical load factors of beams. Stability analysis is performed on the beams according to ultimate limit state (ULS). Live load is 4 kN/m2 and permanent load is 0.3 kN/m2. They are applied as line load to the middle of the top flange of secondary beams. Mesh size of beams is 30mm and mesh size of steel grating is 100mm (see example in Figure 18).

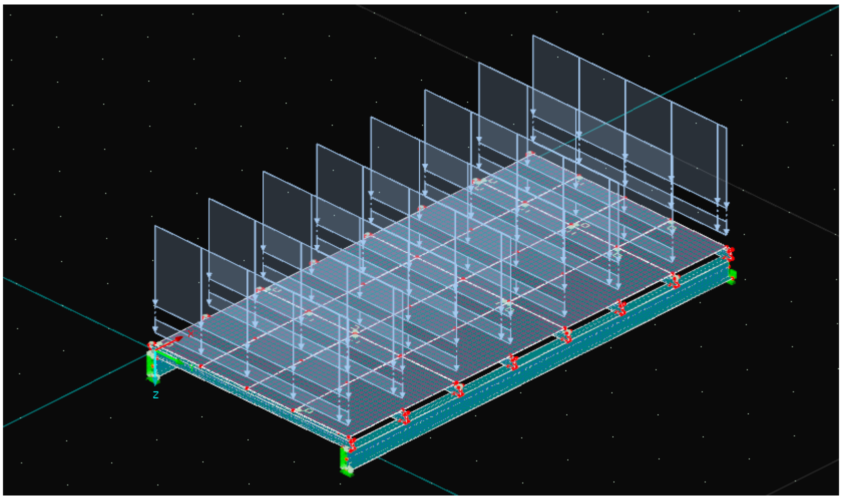


Figure 18. RFEM model Beam 1-IPE360 /Beam 2-IPE160 with loads in ULS design (example)

Twenty-five different buckling modes were calculated in stability analysis as shown in Table 10.

Table 10. RF-Stability results of RFEM model Beam 1-IPE360 /Beam 2-IPE160 (example)

E-Value No.	A Critical Load Factor f [-]	B Magnification Factor α [-]	C Message
1	2.026	1.974	
2	3.388	1.419	
3	4.086	1.324	
4	4.442	1.291	
5	4.456	1.289	
6	4.718	1.269	
7	6.329	1.188	
8	6.373	1.186	
9	7.149	1.163	
10	7.229	1.161	
11	7.370	1.157	
12	7.603	1.151	
13	7.656	1.150	
14	7.790	1.147	
15	7.831	1.146	
16	7.913	1.145	
17	7.989	1.143	
18	8.547	1.133	
19	9.064	1.124	
20	9.962	1.112	
21	10.525	1.105	
22	10.546	1.105	
23	10.637	1.104	
24	11.036	1.100	
25	11.143	1.099	

The result of stability analysis contains both global and local buckling modes of the beams and gratings. The local modes can be shear buckling modes and local flange buckling. But for this research, the main interest is to find the lowest global buckling load for the primary beam and the secondary beam. In the first buckling mode, which corresponds to the smallest critical load factor mode in Table 10, the displacement of

secondary beam already occurred according to the colourful wave in Figure 19 (Figure 19 shows the relative deformation of the beam),
which means that when the critical load factor reaches 2.026, secondary beams start to buckle. Therefore, the rest of 24 modes are not important for secondary beams in this model. The critical load factor for secondary beams in this model is 2.2026. However, in this mode, there is no displacement of the primary beam. Then the rest 24 modes will be checked for the primary beam. Same as the secondary beam, in the second buckling mode (critical load factor is 3.388), primary beams start to buckle according to Figure 20, so the critical load factor for the primary beam is 3.388.

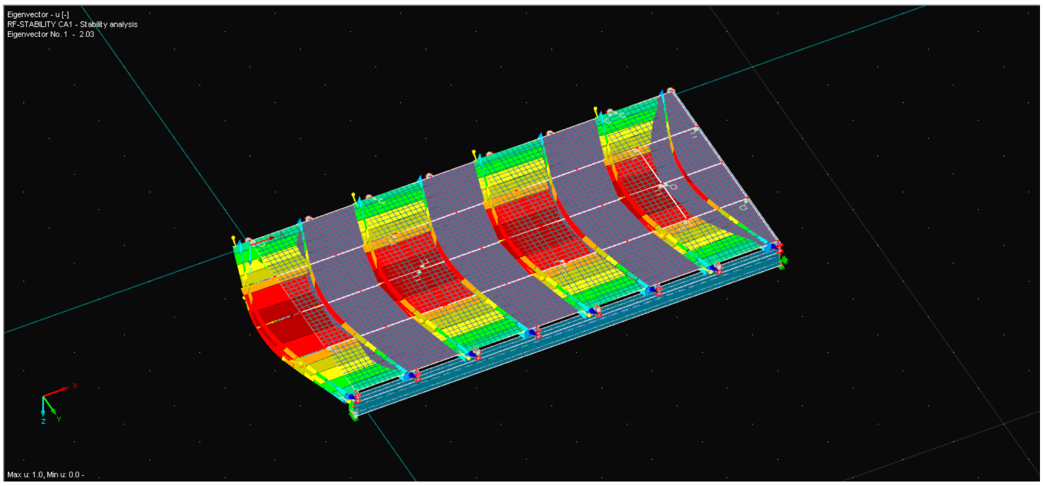


Figure 19. Stability analysis of RFEM model Beam 1-IPE360 /Beam 2-IPE160 in the 2.2026 critical load factor mode for secondary beam (example)

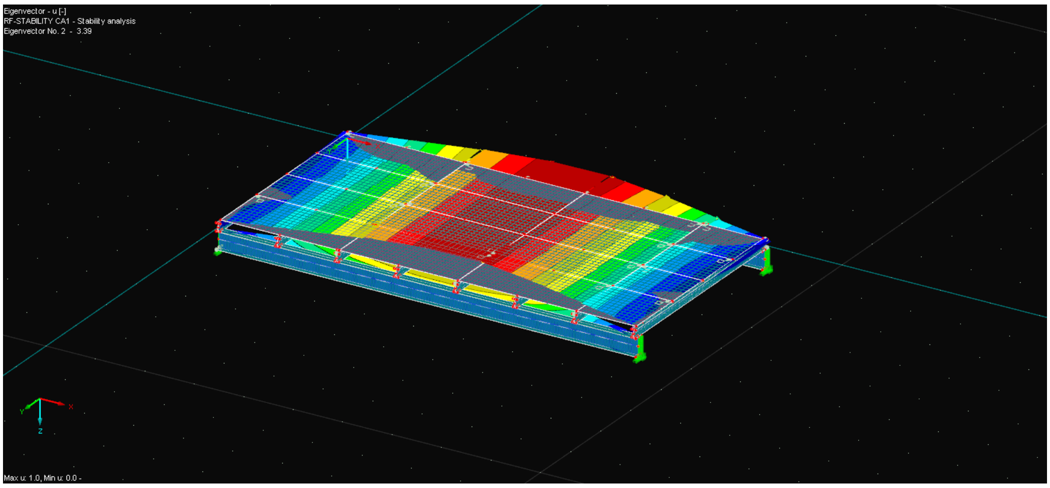


Figure 20. Stability analysis of RFEM model Beam 1-IPE360 /Beam 2-IPE160 in 3.388 critical load factor mode for primary beam (example)

The process of RFEM model building is finished after the critical load factors of secondary and primary beams are found.

3.8 Calculation of the elastic critical buckling moment

After the critical load factors of beams are calculated in RFEM, the next step is to calculate the elastic critical buckling moments of beams. The elastic critical buckling moment is equal to the critical load factor multiplied by the bending moment of the beam as shown in Figure 21.

Critical buckling moment

Maximum load	$M_{Ed} := 126.57 kN \cdot m$
critical load amplifier	$\alpha_{cr.op} := 3.388$
lateral torsional buckling load acc. fem:	$M_{cr} := \alpha_{cr.op} \cdot M_{Ed} = 428.819 \cdot kN \cdot m$

Figure 21. Part of Mathcad of calculating the elastic critical buckling moment in case Beam 1-IPE360 /Beam 2-IPE160 ------Beam 1-IPE360 (critical load amplifier = critical load factor; M_{Ed} = bending moment)

Bending moment of the beam can be calculated easily by building simple model in RFEM as shown in Figure 22.

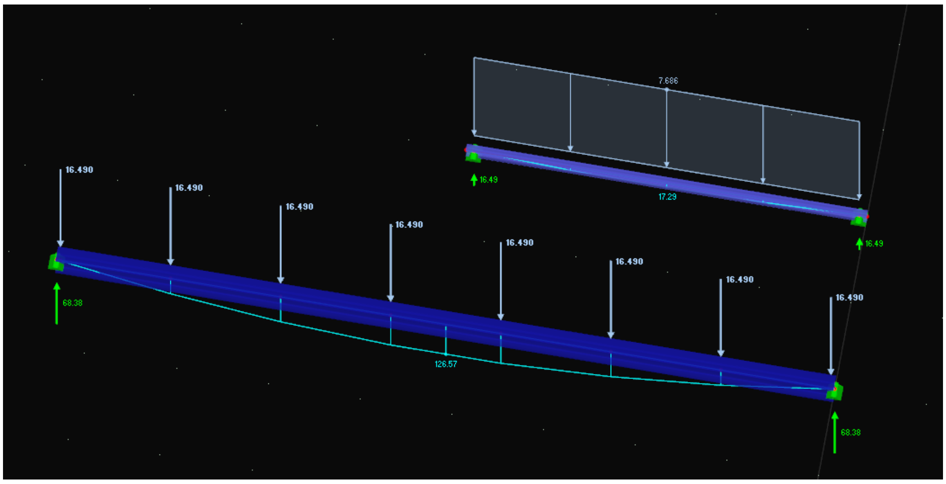


Figure 22. RFEM model of calculating bending moment for Beam 1-IPE360 /Beam 2-IPE160 (example)

4 CALCULATION OF THE EFFECTIVE BUCKLING LENGTH OF THE BEAM

The Eurocodes are design standards, not design handbooks. They omit some design guidance which is considered to be readily available in textbooks or other established sources. It is also accepted that they cannot possibly cover everything that will be needed when carrying out a design. The Eurocode format allows so-called non-contradictory complementary information (NCCI) to be used to assist the designer when designing a structure to the Eurocode. (SCI, 2006, p.5)

According to the introduction, the elastic critical buckling moment of a doubly symmetric cross-section beam can be calculated by using the formula (1):

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(kL)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(kL)^2 G I_t}{\pi^2 E I_z} + \left(C_2 z_g\right)^2} - C_2 z_g \right\}$$

where
E = Young modulus (E = 210000 N/mm^2)
G = Shear modulus (G = 80770 N/mm^2)
I_z = Second moment of area about the weak axis
I_t = Torsion constant
I_w = Warping constant
L = Beam length between points which have lateral restraint
k = Effective length factor which is related to the restrain against lateral bending at the boundaries
k_w = Effective length factor which is related to the restrain against warping at the boundaries
z_g = Distance between the point of load application and the shear centre
C_1= Factor that account for the shape of the moment diagram
C_2 = Factor that account for the point of load application in relation to the shear centre
(NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU)

Normally, the effective buckling length L_{cr} of the beam will be defined firstly based on supporting conditions of the beam to get critical buckling moment. But this formula can also be used reversely: calculate the critical buckling moment first, then get the effective length of beam. The formula is derived from the buckling theory. So, to use the formula properly, there are some extra calculations based on Eurocode required. The commercial programme Mathcad is used to perform these calculations.

The first step is to define the cross-section properties of the beam: height of the beam, widths of the flange and web, torsion constant etc. The second step is to define the elastic critical lateral-torsional buckling moment M_{cr} using RFEM analysis (this step was discussed already in Chapter 3). After these two steps, Mathcad can be used to solve L_{cr} by texting the formula in the programme and solve the undefined variables in the formula.

The effective lateral-torsional buckling length of the beam L_{cr} is calculated with the assumption that the load is applied at the centre line of the beam. If M_{cr} is calculated firstly to define L_{cr}, when the load is on the centre line of the beam, L_{cr} should be longer than the situation that load is at top of the beam, which means this assumption provides safer result.

According to NCCI SN003a-EN-EU (NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU), in the common case of normal support conditions at the ends (fork supports), k and k_w are taken equal to 1 when the transverse load is applied in the shear centre, $C_2*z_g = 0$. However, the formula (1) is applied when the conditions of restraint at each end are at least:

1. restrained against lateral movement
2. restrained against rotation about the longitudinal axis.

Factor C_1 depends on the shape of the moment diagram. But in this situation, the grating platform provides horizontal support for the beam, so there is no specific suggested value for C_1.

On the one hand, if the total length of the beam is divided into segments, in the worst segment, the moment is almost linear (not accurate). At the same time, the load location is assumed to be at the centre line of the beam. Therefore, the value of C_1 can be taken from the condition when the member has concentrated moment applied at the ends as shown in Figure 23.

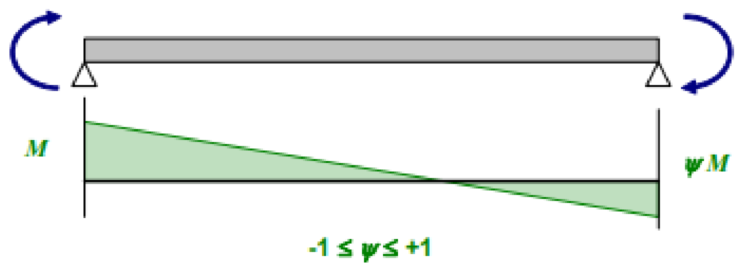


Figure 23. Member with end moments (NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU)

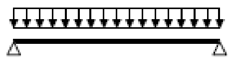
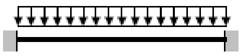
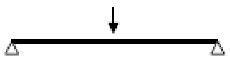
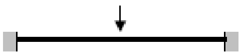
Because the load is evenly distribution load in each segment, ψ should be +1.00 in this situation. According to Table 11, in NCCI, when ψ is +1.00, C_1 should be taken as 1.00.

Table 11. Value of C1 for constant moment and moment distribution for evenly distributed loading (for k=1) (NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU)

ψ	C_1
+1,00	1,00
+0,75	1,14
+0,50	1,31
+0,25	1,52
0,00	1,77
-0,25	2,05
-0,50	2,33
-0,75	2,57
-1,00	2,55

On the other hand, if the moment distribution is considered for the whole beam, then the moment diagram of the beam is close to the case of a member loaded by transverse loading and pinned supports as shown in Table 12.

Table 12. Values of factors C1 and C2 for cases with transverse loading (NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU)

Loading and support conditions	Bending moment diagram	C_1	C_2
		1,127	0,454
		2,578	1,554
		1,348	0,630
		1,683	1,645

Note : the critical moment M_{cr} is calculated for the section with the maximal moment along the member

In this case, the value of factor C_1 should be taken as 1.127.

However, in this research, the rest of chapters (except Chapter 6) will only focus on the situation when $C_1=1$. There is additional appendix (Appendix 5) which shows the results of the narrow flange primary I-beam for both $C_1=1$ and $C_1=1.127$. It is necessary to mention that both of the results are correct if the same values of C_1 and C_2 are used to calculate the critical buckling moment of the beam by using the effective length factor found as a result in this research.

Hence, the formula (1) can be simplified for a symmetric I-shaped steel beam with $C_1=1$ and $C_2*z_g = 0$. And the simpler formula (3) is applied in Mathcad to solve L_{cr}.

$$M_{cr} = \frac{\pi^2 EI_z}{L_{cr}{}^2}\left\{\sqrt{\frac{I_w}{I_z} + \frac{L_{cr}{}^2 GI_t}{\pi^2 EI_z}}\right\} \qquad (3)$$

For Mathcad calculation process example, please check Appendix 4.

5 RESULTS OF NARROW FLANGE PRIMARY I-BEAMS

After all the model building and calculations are done, results of the effective lateral-torsional buckling length of the narrow-flange primary I-beam are collected and organized into Tables 13-15:

Table 13. Results of the effective buckling length of narrow flange primary beams when the profile of the secondary beam is IPE140 (width of platform=2m)

	L(m)	L_{cr} (m)	FACTOR ($k_{LT} = L_{cr}/L$)
IPE200	6.073	1.841	0.30
IPE270	8.473	2.570	0.30
IPE360	10.873	3.557	0.33

Table 14. Results of the effective buckling length of narrow flange primary beams when the profile of the secondary beam is IPE160 (width of platform=4.2m)

	L (m)	L_{cr} (m)	FACTOR ($k_{LT} = L_{cr}/L$)
IPE240	6.082	2.000	0.33
IPE360	8.482	3.196	0.38
WI400-5-15*220	10.882	5.295	0.49

Table 15. Results of the effective buckling length of narrow flange primary beams when the profile of the secondary beam is IPE220 (width of platform=6.2m)

	L (m)	L_{cr} (m)	FACTOR ($k_{LT} = L_{cr}/L$)
WI400-5-12*220	8.510	4.669	0.55
WI450-5-16*250	10.910	6.027	0.55

In Tables 13-15, L is the full length of the beam; L_{CR} is the effective lateral-torsional buckling length; k_{LT} is the effective length factor. And these results are based on factor C_1= 1. There are also results for C_1=1.127 in Appendix 5 for references. The differences between the two sets of results are small (maximum 7%).

The regulations of the values in the results are clear:

1. In each table, when the length of platform becomes bigger, the profile of the primary beam will be bigger relatively, and the effective length factor will be bigger.
2. Generally, when the size of platform is bigger (width and length are both bigger), the profiles of the secondary beam and the primary beam will be bigger relatively, and the effective length factor will be bigger.

The effective buckling length of the primary beam is decreased because of two restraining factors:

1. horizontal stiffness from steel gratings
2. torsion restraints from secondary beams (end stiffness of the beam and the stiffness of connection)

Theoretically, when the size of the grating platform is bigger, there is more horizontal stiffness from gratings, but stiffness is also related to the length of the primary beam. When the length of the secondary beam is the same, but the primary beam is longer, the relative lateral-torsional stiffness of the primary beam itself decreases (if ratio h/b in the beam is the same). And when the length of the secondary beam increases, changes of the secondary beam end stiffness depend on the span of beam. End stiffness of the secondary beam can be described with following formula:

$$\frac{3EI}{L} = M \qquad (4)$$

where
E= Elastic modulus(N/mm^2)
I = Second moment of inertia (mm^4)
L = Span of the beam (mm)
M = Elastic moment (N*mm) at beam end
(M=The moment is required to rotate the end of beam through a unit angle.)

In case the size of the secondary beam increases (due to longer span of primary beam), the stiffness of connection to the primary beam also increases because of the changes in flange thicknesses and the geometry in the connection. But because the bolts are the same in each secondary beam and primary beam connection, the increase of stiffness of the connection can be less than the increase of end stiffness of the secondary beam. In certain situations, the limited stiffness of connection can restrict restraining effect provided by the secondary beams.

For wider platforms, it can be assumed that: even though the grating panel has a higher horizontal stiffness, and it tries to prevent primary beams from lateral movement, the torsional restraints (due to secondary beams) for the primary beam in relation to beam’s own torsional stiffness is decreasing because of changes in the secondary beam and in connection stiffness. It is quite obvious that the rotational restraint provided by the secondary beams and the connection between the secondary beam and the primary beam plays an important role in defining lateral-torsional stability of the primary beam.

According to the development of Euler´s formula (Euler buckling cases) (Gere & Timoshenko, 2009, p.57), when both sides of a beam are connected by pinned connections, the effective length factor is 1, but in the results, all the effective length factors are smaller than 0.6 (including the situations when C_1=1.127), so the grating together with the secondary beams does provide quite good restraints to the lateral-torsional buckling of narrow flange primary I-beams.

In the RFEM model, stiffness of the grating panel in global z direction is not defined, and it is only provided by the bolts. At the same time, all the rigid elements between secondary and primary beams are 30mm long for convenient modelling. Therefore, the results are on the safe side, and the real effective length factor can be even smaller than the numbers in the results.

In total, according to the results, it is possible to give suggestions for the steel design of the narrow flange primary I-beam which supports steel gratings with secondary beams:

1. To be precise, the effective length factor of primary beam can be from 0.3 to 0.55 depending on the different size of the area of grating platform. In real steel design, it is suggested to use the results as a reference when choosing the suitable value of factor according to the real situations.
2. In general, to be on safe side, it is suggested to use 0.6 as the effective length factor for the primary beam if the beam span is smaller than 10.8 m and the platform width is smaller than 6.2m.

Please also take the results of the situation when C_1=1.127 (Appendix 5) into consideration during steel design. It is important that when applying the results of this research to the calculation of the critical buckling moment of a beam by simple analytical formulas, same values of C_1 and C_2 for calculating the effective buckling length should be taken.

6 EFFECTS OF THE EFFECTIVE LENGTH FACTOR ON THE MOMENT CAPACITY

From the previous chapter, it is clear that steel grating platform can reduce the effective length factor of narrow flange primary I-beams. But in steel beam design, generally all the values and calculations will in the end affect the design moment resistance which is one of the most significant values for steel beams. Apparently, if steel gratings can improve the stability of the beam in relation to lateral torsional buckling, the design buckling resistance moment should be bigger than in the situation when there are no effects from gratings. The design buckling resistance moment can be calculated based on Eurocode.

According to Eurocode 3, the design buckling resistance moment of a lateral unrestrained beam is calculated by the following formula:

$$M_{b,Rd} = \chi_{LT} W_y \frac{f_y}{\gamma_{M1}} \qquad (5)$$

where
W_y = Appropriate section modulus as follows:

- $W_y = W_{pl,y}$ for Class 1 or 2 cross-sections
- $W_y = W_{el,y}$ for Class 3 cross-sections
- $W_y = W_{eff,y}$ for Class 4 cross-sections

χ_{LT} = Reduction factor for lateral-torsional buckling
(Eurocode 3: Design of steel structures - Part 1-1: General rules, 2005)

The formula indicates a linear relationship between the reductions factor and the design buckling resistance moment, which means that the increase of this reduction factor represents the increase of the capacity of moment.

The reduction factor is related to the value of non-dimensional slenderness. Their relationship can be expressed by following formulas:

$$\chi_{LT} = \frac{1}{\Phi_{LT} + \sqrt{\Phi_{LT}^2 - \bar{\lambda}_{LT}^2}} \qquad (6)$$

$$\Phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0{,}2\right) + \bar{\lambda}_{LT}^2\right] \qquad (7)$$

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}} \qquad (8)$$

where

$\chi_{LT} \leq 1$
α_{LT} = Imperfection factor
M_{cr} = Elastic critical moment for lateral-torsional buckling

The relationship between the reduction factor and the non-dimensioned slenderness can also be plotted as graph (see Figure 24):

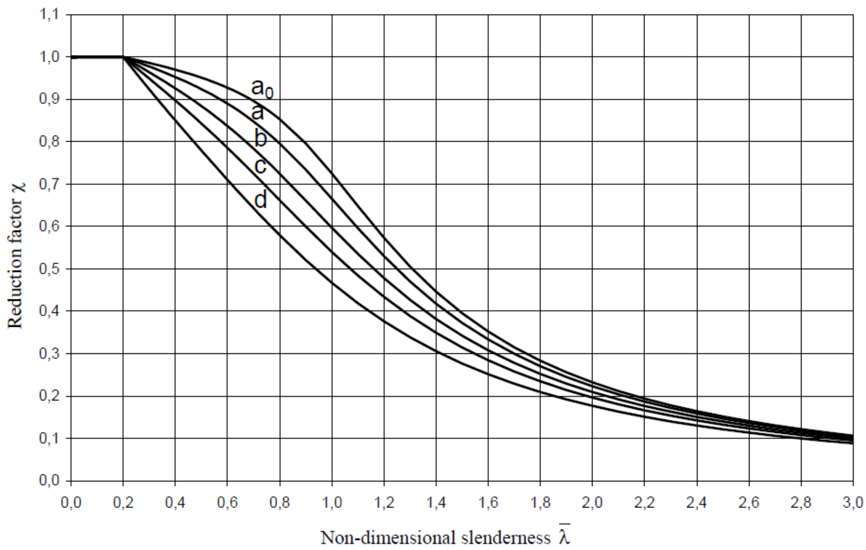


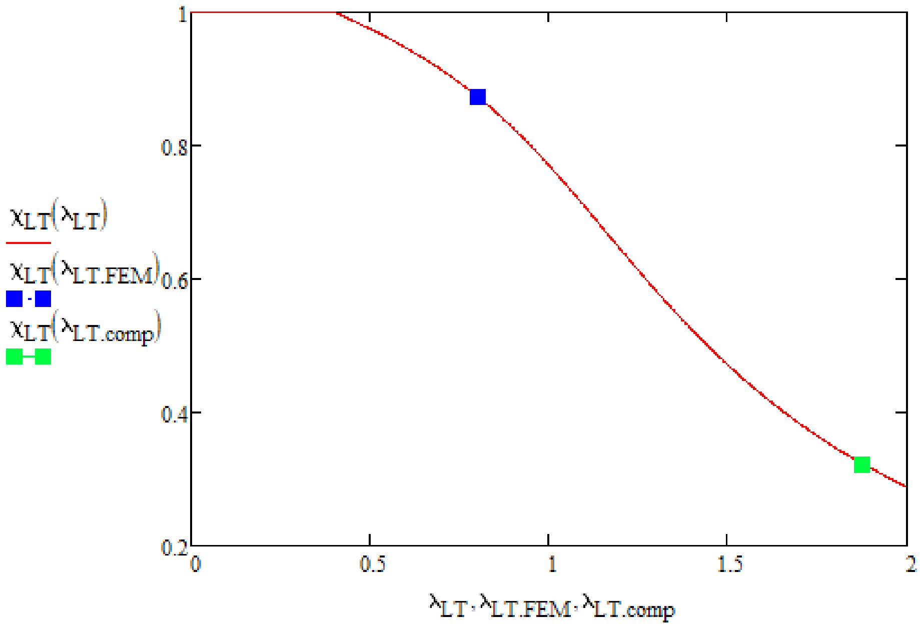
Figure 24. Values of the reduction factor for the appropriate non-dimensional slenderness (Eurocode 3: Design of steel structures - Part 1-1: General rules, 2005)

For primary beams in RFEM models, the elastic critical moments are calculated in Chapter 4 by a critical load factor and applied bending moment (Appendix 4), so it is easy to calculate the non-dimensional slenderness and the reduction factor according to Figure 27. If gratings and secondary beams do not affect the primary beam at all, then the effective buckling length should be the full length of the primary beam. Hence, the critical buckling moment can be calculated using the formula (1). In this case, it must be calculated with assumption: 1. load is applied at the top flange; 2. uniform load distribution(C_1=1.127). The non-dimensional slenderness and the reduction factor in this situation can be calculated in the same way. Effects of the decrease of the effective length factor on the capacity of moment can be shown by comparing these two reduction factors. For detailed Mathcad calculation process, please check Appendix 6.

Two cases are chosen to roughly define the range of how the capacity increases:

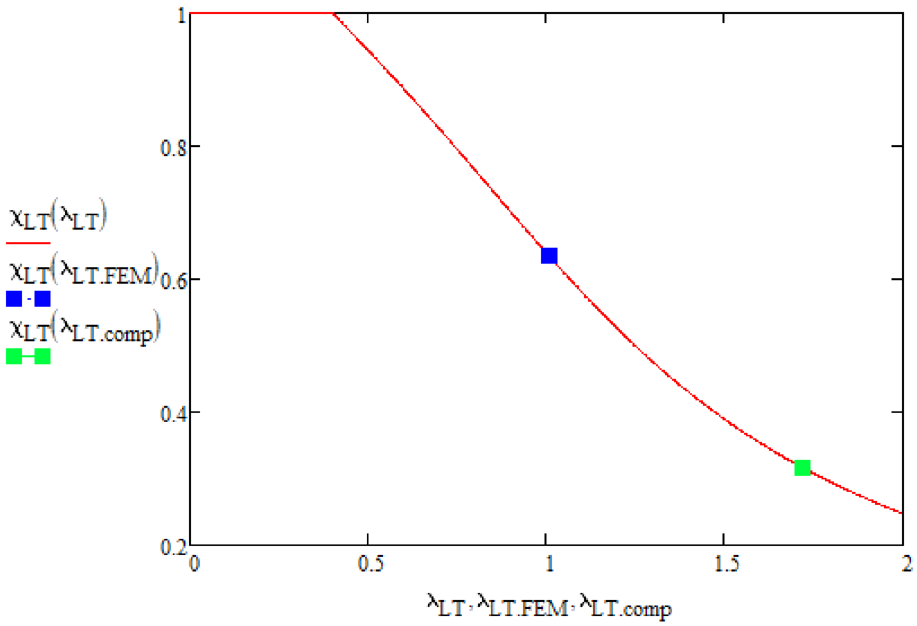
Case 1: Beam 1-IPE200 /Beam 2-IPE140
Case 2: Beam 1- WI450-5-16*250 /Beam 2-IPE140

These two cases are the extreme cases in RFEM models: the smallest effective length factor is calculated from Case 1 and the biggest effective factor is calculated from Case 2. The results of these two cases are shown in Figures 25 and 26.



$$\frac{\chi_{LT}(\lambda_{LT.FEM})}{\chi_{LT}(\lambda_{LT.comp})} = 2.708$$

Figure 25. The effects of the decrease of effective length factor on the capacity of moment of case Beam 1-IPE200 /Beam 2-IPE140



$$\frac{\chi_{LT}(\lambda_{LT.FEM})}{\chi_{LT}(\lambda_{LT.comp})} = 2.005$$

Figure 26. The effects of the decrease of effective length factor on the capacity of moment of case Beam 1- WI450-5-16*250 /Beam 2-IPE140

In Figures 25 and 26, the blue point is the reduction factor χ_{LT} calculated from the RFEM model and the green point is the reduction factor when the lateral-torsional buckling length equals the span of the beam. The red line is the relationship between reduction factor and non-dimensional slenderness according to different cross-section classes of the beam. The ratio of two reduction factors shows how much the moment capacity is increased. According to Figures 25 and 26, the moment capacity increases around 2 to 2.7 times comparing to the beam without any lateral support along its span.

In a more general way, the relationship between the effective length factor and the capacity of design buckling resistance moment can be calculated directly by using the same Excel sheet (Appendix 1 and 2) when defining profiles of the beams.

Beam WI450-5-16*250 is chosen as an example of calculation. Ten different effective length factors have been taken to calculate the design buckling resistance moments. When there are no effects from gratings, the effective length of lateral-torsional buckling should be the full length of the beam. By comparing the moment values calculated for beams having smaller effective lengths with the ones calculated for beams having full effective lengths, it is possible to define the relationship between the effective length factor and the capacity of moments (see Figure 27).

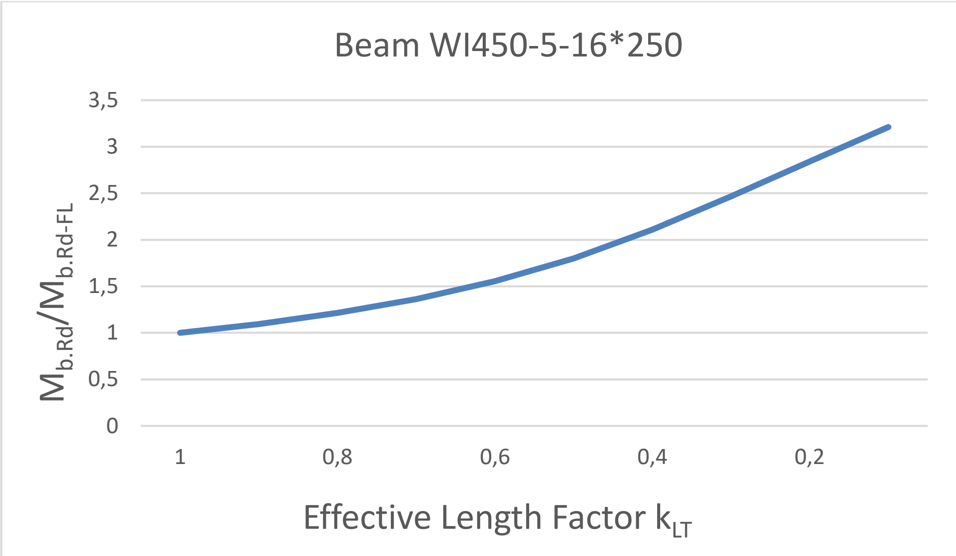


Figure 27. Relationship between effective length factor and the change of capacity of buckling resistance moment

In the graph, $M_{b.Rd}$ is the calculated design buckling resistance moment with different effective length factors. and $M_{b.Rd\text{-}FL}$ is the moment when the effective length factor of the beam is 1. The shape of the curve can be different due to the cross-section of the beam and the applied load as shown in Table 16. This is just an example of a narrow-flange I-beam (h/b=1.8) under evenly distribution load in the c group buckling curve.

Table 16. Recommended values for lateral torsional buckling curves for cross-sections using equation (Eurocode 3: Design of steel structures - Part 1-1: General rules, 2005)

Cross-section	Limits	Buckling curve
Rolled I-sections	$h/b \leq 2$ $h/b > 2$	a b
Welded I-sections	$h/b \leq 2$ $h/b > 2$	c d
Other cross-sections	-	d

7 SENSITIVITY TEST

In the RFEM model, the secondary beams support the full area of the grating platform. But in real life, things can be different: the last secondary beam only supports part of the grating and the primary beam is separated into two beams because of the location of column or support (see Figure 28). Therefore, there is one sensitivity test for the results: what if the secondary beam is 0.6m away from the end of the primary beam?

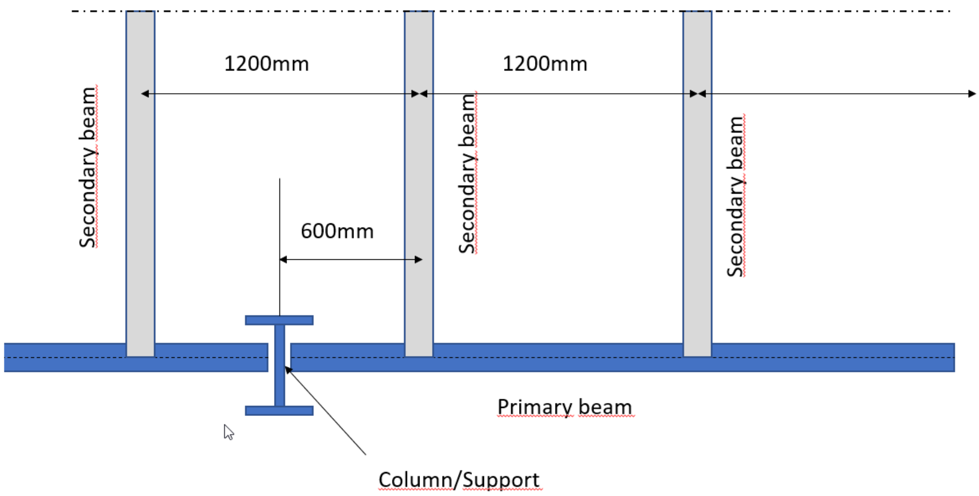


Figure 28. Demonstration of sensitivity test

Because the results of Beam 2-IPE220 are bigger, which means that gratings provide less effects on the primary beams, and they are more sensitive to the change of gratings, the sensitivity tests are only applied to the two models of Beam 2-IPE220. To be safer, all the secondary beams were moved 0.6m along the primary beam, and one grating panel was deleted (see example in Figure 29).

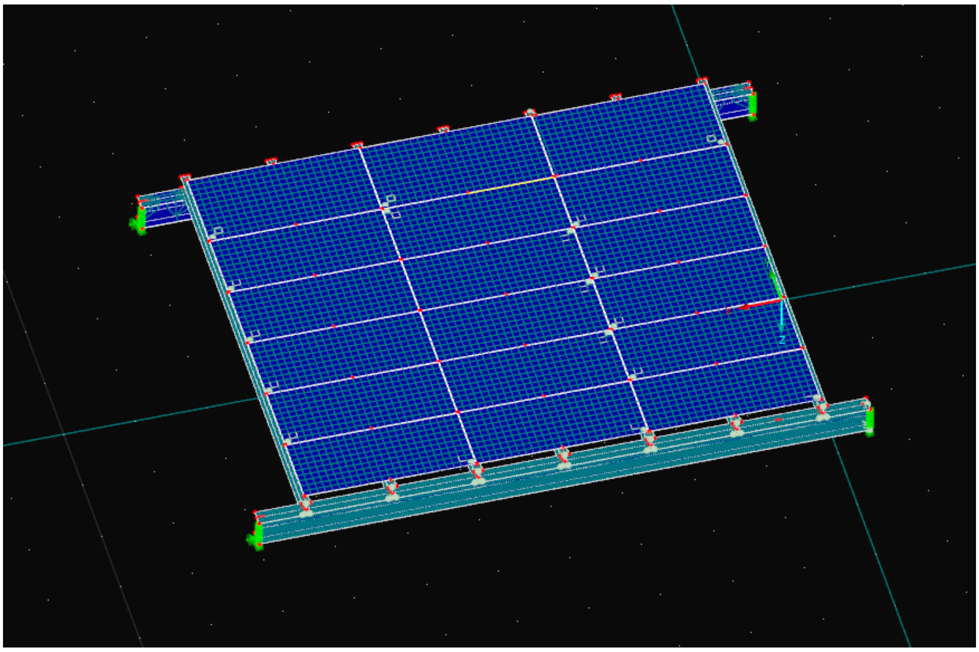


Figure 29. Sensitivity test for RFEM model Beam 1-WI400-5-12*220/Beam 2-IPE220 (example)

The results of sensitivity test are shown in Table 17:

Table 17. Results of sensitivity tests

TEST	L (m)	L_{cr} (m)	FACTOR (($k_{LT} = L_{cr}/L$)
WI400-5-12*220	8.510	4.909	0.58
WI450-5-16*250	10.910	6.231	0.57

As shown in Table 17, the difference between the results obtained from the sensitivity tests and the models used for research is very small (5%). Therefore, the suggestion of effective length factor (0.6) for the narrow flange primary I-beam is valid, and this value allows minor modifications in the configuration of the grating platform. (However, according to Appendix 5, when factor C1=1.127, the effective length factor is bigger than 0.6 in sensitivity test. This should be noticed)

8 RESULTS OF SECONDARY BEAMS

Following the same process of calculating the primary beams, the effective lengths of secondary beams can also be calculated. But there are already good studies about how steel gratings affect the buckling length of the secondary beams, for example, Stabilisierung von I-Trägern durch Gitterroste (Gilde, 2003). As a reference, the results in this German report have been compared with the ones which are calculated in the models. Tables 18, 19 and 20 below show the results of calculations.

Table 18. Results of the effective buckling length of the secondary beam IPE140 with different primary beams

	L (m)	L_{cr} (m)	FACTOR (L_{cr}/L)	L_{cr} (m) (German)	FACTOR ($k_{LT} = L_{cr}/L$) (German)
IPE140-IPE200	2.100	3.407	1.62	1.526	0.73
IPE140-IPE270	2.135	3.294	1.54	1.526	0.71
IPE140-IPE360	2.170	3.238	1.49	1.526	0.70

Table 19. Results of the effective buckling length of the secondary beam IPE160 with different primary beams

	L (m)	L_{cr} (m)	FACTOR (L_{cr} /L)	L_{cr} (m) (German)	FACTOR (($k_{LT} = L_{cr}$ /L) (German)
IPE160-IPE240	4.320	2.285	0.53	2.352	0.54
IPE160-IPE360	4.370	2.158	0.49	2.252	0.54
IPE160-WI400-5-15*220	4.420	2.152	0.49	2.252	0.51

Table 20. Results of the effective buckling length of the secondary beam IPE220 with different primary beams

	L (m)	L_{cr} (m)	FACTOR (L_{cr} /L)	L_{cr} (m) (German)	FACTOR (($k_{LT} = L_{cr}$ /L) (German)
IPE220-WI400-5-12*220	6.420	3.668	0.57	3.878	0.60
IPE220-WI450-5-16*250	6.420	3.606	0.56	3.878	0.60

Apart from the result in Table 18, the difference between the two sets of results is only 2-10%. Also, the regulations of the values of the results are the same. But in Table 18, the results are not reasonable. It might be because the secondary beam is really short and there are only fastenings at middle and ends of the beam. Another reason might be that the stiffness of grating in global z direction is not defined accurately, and it affect a lot of the effective length of secondary beams. To prove the assumptions, the bolts between secondary beams and primary beams were changed into M10 and M5. The results of this test show that the size of the bolt does not affect much of results obtained for the primary beams, but it affects the ones for the secondary beams, especially if the span of the secondary beam is short. However, it is not possible to define the stiffness of grating in global z direction properly. Therefore, this model is more suitable for calculating the primary beams. As for the secondary beam, it is suggested to use the formulas in the German research. (Gilde, 2003)

9 TESTS OF WIDER FLANGE PRIMARY I-BEAMS

In all the models, the ratio of the height and width of the primary beam is 1.8 or more than 1.8. But what if the flange of the primary beam is not that “narrow”? Will the results of effective length of the primary beam change a lot and what is the direction of the change? To solve these questions, four models with wider flange primary beams were built using the same procedure as for the earlier models. The profiles of wider flange primary beams are shown in Table 21. Also, for these wider beams, the deflection of the beam is critical for the design. The ratio of height and width of the wider flange I-beams is designed on purpose in different range of numbers.

Table 21. Beam profiles for different platform configuration for wide-flange primary I-beams

Span	**B = width of platform (m)**			
L (m)	6,2		6,2	
	beam 1(wide)	beam 2	beam 1(m)	beam 2
6,0				
8,4	WI300-5-15*300	IPE220	WI360-5-12*230	IPE220
10,8	WI370-5-20*330	IPE220	WI410-5-18*270	IPE220

And the results of wider flange primary I-beams are shown in Table 22:

Table 22. Results of the effective buckling length of wider flange primary beams when the profile of the secondary beam is IPE220 (width of platform=6.2m)

	L (m)	L_{cr} (m)	FACTOR ($k_{LT} = L_{cr} /L$)
WI300-5-15*300	8.510	6.803	0.80
WI370-5-20*330	10.910	7.928	0.73
WI360-5-12*230	8.510	4.723	0.55
WI410-5-18*270	10.910	6.853	0.63

The regulation of the values in Table 22 are quite interesting. Even though the sizes of platforms are different, their effective length factors are more related to the ratio of height and width of the beam. Thus, if the results of wide-flange primary beams are arranged by the ratio of height and width of the profile, Table 22 can be changed into Table 23:

Table 23. Results of the effective buckling length factor of wider flange primary beams ranking by the ratio of height and width of the beam

	h/b	FACTOR ($k_{LT} = L_{cr} /L$)
WI360-5-12*230	1.56	0.55
WI410-5-18*270	1.51	0.63
WI370-5-20*330	1.12	0.73
WI300-5-15*300	1.00	0.80

If we also take narrow flange I -beams into consideration, then the results of effective length factor can be shown in this chart in Figure 30 below (only the biggest result in each range is taken):

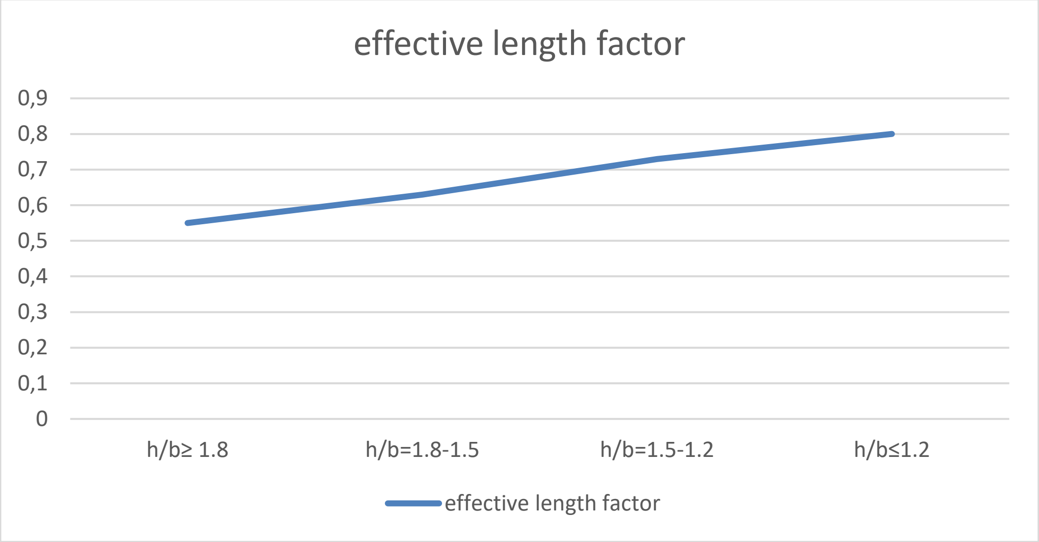


Figure 30. chart of the relationship between the ratio of height and width and the effective length factor of beams

It is obvious that when the height to width ratio of the beam is smaller, the effects from grating on the lateral-torsional buckling of beams are smaller. And the relationship between the height and width ratio of the beam and the effective length factor can roughly be (if the area of platform on two primary beams is smaller than 6.2mX10.8m and C_1=1):

1. when h/b is 1.8 or more than 1.8, the effective length factor is smaller than 0.6
2. when h/b is from 1.8 to 1.5, the effective length factor is smaller than 0.7
3. when h/b is from 1.5 to 1.2, the effective length factor is smaller than 0.8
4. when h/b is 1.2 or less than 1.2, the effective length factor is bigger than 0.8

These results are reasonable because wider flange primary beam has a higher torsional and horizontal stiffness. The additional restraint effect of grating platform is rather small in case of wider flange primary beams. However, these results are based on a few testing models, and a grating platform with wider flange beams requires more investigations.

10 EXTRA STIFFNESS TESTS OF CONDITION 3 CONNECTION

In all the RFEM models, the secondary beams are on the top of the primary beams and connected by two bolts in bidiagonal direction. It has been assumed that condition 2 and condition 3 connections are stiffer than the condition 1. Because of the strict attitude of science, the stiffnesses of condition 3 connections are tested by using the programme IDEAStatiCa.

For the models with IPE140 or IPE160 as the profile of the secondary beam, there is much higher stiffness in condition 3 connections than in the condition 1 connections. But for the models with the secondary beam IPE 220, when the profile of the primary beam is WI450-5-16*250, there is lower stiffness in condition 3 connection than in the condition 1. The reason is assumed to be that web slenderness(h_w/t_w) of the primary beam is too high. To prove this assumption, one test was performed in IDEAStatiCa: changing the profile of the primary beam from WI450-5-16*250 to WI450-6-15*250. The results show that when the primary beam is WI450-6-15*250, the condition 3 connection is almost as stiff as the condition 1 connection. In the beam WI450-5-16*250, the web slenderness h_w/t_w is 83.6, but in the beam WI450-6-15*250, the web slenderness h_w/t_w is 70. Apparently, more tests are needed to draw a clear conclusion. However, it can be roughly suggested that when applying the results of this research into steel design, the stiffness of connections between secondary beams and primary beams need to be checked in condition 1 and condition 3 if the web slenderness of the primary beam is more than 70.

11 CONCLUSION

Based on the results of this research on the effective buckling length of narrow flange primary I-beams, steel grating does provide adequate restraints to the lateral-torsional buckling of primary beams. The results suggest that 0.6 is a safe effective length factor for the narrow flange primary I-beam when the area of platform on two primary beams is smaller than 6.2mX10.8m, and this value can be smaller according to the size of grating platform(for both situations: C_1=1 and C_1=1.127). In steel design, engineers can use Tables 13-15, results of sensitivity tests and Appendix 5 as references to choose the best effective length factor for real situations. Also, it is important that when applying the results in this research to the calculation of the critical buckling moment of a beam by simple analytical formulas, the same values of C_1 and C_2 as calculating the effective buckling length should be taken. Due to the decrease of the effective length factor k_{LT} of the primary beam, the capacity of design buckling moment for the narrow flange primary I-beam can be increased around 2 to 2.7 times (if C_1=1.127) comparing to the situation when the effective length factor of the beam is 1. This result shows the power that the effective length factor has on the moment capacity of the beam. It can be used as a reference.

But it is still suggested to use the found results of the effective length factor during steel design.

According to the tests of wider flange primary I-beams, it is shown that the effective buckling length of the primary beam is also related to the ratio of height and width of the beam. It is roughly suggested that (if the area of platform on two primary beams is smaller than 6.2mX10.8m and C_1=1):

1. when h/b is 1.8 or more than 1.8, the safe effective length factor is 0.6.
2. when h/b is between 1.8 to 1.5, the safe effective length factor is 0.7.
3. when h/b is between 1.5 to 1.2, the safe effective length factor is 0.8.
4. when h/b is 1.2 or less than 1.2, grating platform does not provide remarkable restraint for the main beam regarding lateral-torsional buckling.

More investigation of wider flange I-beams is required to support this conclusion.

It is also important to notice that the type of the secondary beam to the primary beam connection as well as web slenderness of the primary beam may provide a remarkable effect on restraining primary beams from lateral-torsional buckling. Further tests with different kind of connections are needed to get better knowledge on this topic.

REFERENCES

Access/Stainless Steel Grating - Flooring, Platforms, Access Systems. (n.d.). Retrieved 29 February 2020 from http://www.webforge.com.my/access/stainless-steel-grating

Eurocode 3: Design of steel structures - Part 1-1: General rules. (2005).

Fixings. (n.d.). Retrieved 29 February 2020 from http://www.lichtgitter.com.tr/elocal/bef-mat.html

Gere, J. M. (2001). *Mechanics of Materials,* 5th Edition. Thomson.

Gere, J. M., & Timoshenko, S. P. (2009). *Theory of Elastic Stability,* 2nd Edition. New York.

Gilde, H. (2003). *Stabilisierung von I-Trägern durch Gitterroste.* Regensburg.
(2009-04-01). *International standard ISO 898-1.*

NCCI: Elastic critical moment for lateral torsional buckling SN003a-EN-EU.

SCI. (2016). *Steel Designer's Manual.* John Wiley and Sons Ltd.

Appendix 1

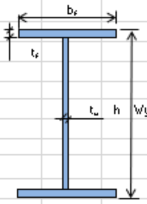
Calculation of the profile of beam in case: Beam 1-IPE360 /Beam 2-IPE160 ------Beam 2-IPE160 (example)

SWECO **DESIGN OF SINGLE SPAN STEEL MEMBER TO EN 1993-1-1** DATE **18.2.2020**

SYMMETRIC I -SECTION

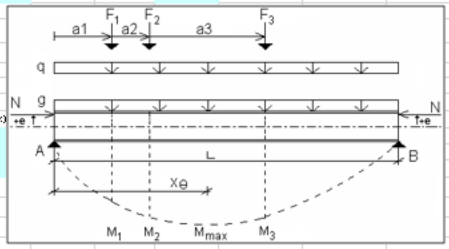
PROJECT:	Insinöörityö	ROLLED PROFILE:	IPE160	ROLLED EUROPEAN STEEL SHAPE
AREA	Esimerkkitaso B=4.2 m	WELDED BEAM:		H-t_w/$t_f$xb (depth-web/t,flange x b,flange) (mm)
BEAM	BEAM2	BEAM WEIGTH	66 kg	

Steel member type: **BEAM**



			h =	160	mm	STEEL GRADE:	**S235**	GROSS SECTION CLASS:	
It =	3,38E+04	mm4	t_w =	5	mm				
Iw =	3,98E+09	mm6	t_f =	7,4	mm	f_y =	235 N/mm2	web:	1
Iz =	6,83E+05	mm4	b_f =	82	mm	f_u =	360 N/mm2	flange:	1
Wz =	1,666E+04	mm3	r =	9,0	mm	G =	80800 N/mm2		
i_z =	18,4	mm	A =	2009	mm2	E_s =	210000 N/mm2		
G =	15,8	kg/m	Iy =	8,693E+06	mm4	βw =	0,9 factor		
			Wy =	1,087E+05	mm3	ε = $\sqrt{235/fy}$ =	1,000		
PLASTIC PROPERTIES			i_y =	65,8	mm	βω =	1,00		
W_{pl} =	1,24E+05	mm3							
$M_{pl,Rd}$ =	**29,1**	kNm							

LOADINGS ON MEMBER (nominal loads)



(to be filled in numerical order, value = 0, if load doesn't exist)

				Dead load portion (%)	Live load(%)
F1(kN)-	0,00	a1(m)=	0,00	0	0
F2(kN)-	0,00	a2(m)=	0,00	0	0
F3(kN)-	0,00	a3(m)=	0,00	45	0
Fd1(kN)-	0		γ F1=	0	
Fd2(kN)-	0		γ F2=	0	
Fd3(kN)-	0		γ F3=	0	
Axial load N (kN) =	0,00		Dead load portion (%)-	30	

g (DL) (kN/m)=	0,36	gd =	0,59	kN/m	L (m) =	4,20	N_{Ed}(kN)=	0,0	Excentricity e(mm) = 0
q (LL) (kN/m)=	4,8	qd =	7,2	kN/m			Compression is positive		

MOMENT AND SHEAR in ultimate limet state: Load factors: γ G = 1,15 γ Q = 1,5 for distributed loads

Reactions			X_0 =	2,10	m					
ULS R_{AGd}=	1,2	kN	Design values:	**M_{max}= M_{Ed}=**	**17,18**	kNm		R_{BGd}=	1,2	kN
R_{AQd}=	15,1	kN		**V_{Ed}=**	**16,4**	kN		R_{BQd}=	15,1	kN
	16,4	kN	M_{Ed1} =	---	kNm	V_{Ed1}=	--- kN		16,4	kN
SLS R_A =	11,2	kN	M_{Ed2} =	---	kNm	V_{Ed2}=	--- kN	R_B =	11,2	kN (SLS)
			M_{Ed3} =	---	kNm	V_{Ed3}=	--- kN			

DESIGN CRITERIA	γ_{M2} =	1,25		BUCKLING;		DEFLECTION LIMITS:	
Partial safety factors	γ_{M0}, γ_{M1} =	1,00		buckling length(y)=	4,2 m	total deflectio	200
Web stiffener spacing	a =	4200	mm	buckling length(z)=	4,2 m (weak)	live load L	250
Factor / lateral torsional buckling (EC3:6.3.2.3)	$\bar{\lambda}_{LT,0}$ =	0,4	as specified in NA				
Factor / lateral torsional buckling (EC3:6.3.2.3)	β =	0,75	as specified in NA	Load combination in deflection check			
Shear strength/ Tensile yield limit (EC3:5.1)	η =	1,0		g_k+F+ψq_k , ψ factor for variable actions =			1,0

CALCULATION OF ELASTIC CRITICAL MOMENT IN LATERAL BUCKLING — Calculaton method for lateral bucklir General case, 6.3.2.2

k_w=1,0 ja k=1,0 (beam ends are not fixed,= fork support at beam ends) — free span / compression flange = 2,1 m

C_1 = 1,127 equivalent uniform load on beam span — z_g= 80 mm, distance from load point to the centroid of beam

C_2 = 0,454 factor for transverse loading of the beam — + when load on top flange

(C_2 =0 , if bending moment diagram is linear between lateral restraints or transverse load is applied at shear centre of the beam)

COMBINED AXIAL COMPRESSION FORCE AND BENDING:

Factor for equivalent equally distributed moment M_y : C_{my} = 0,95 depends on moment diagram shape on the overall length of beam value 0,4...1,0. see Table B.3 in EN 1993-1-1

ANALYSIS AND DESIGN SUMMARY:

ACTIONS ON STEEL MEMBER			STRENGTH OF CROSS SECTION			STABILITY OF STEEL MEMBER			NOTE ! / ADDITIONAL CHECK
Action	value	unit	Capacity	value	rate E_d/R_d	Capacity	value	rate E_d/R_d	
Moment $M_{Ed,y}$	17,2	kNm	$M_{c,Rd}$	29,1	**0,591**	$M_{yb,Rd}$	20,3	**0,845**	**OK**
Shear V_{Ed}	16,4	kN	$V_{c,Rd}$	98,5	**0,166**	$V_{c,Rd}$	98,5	**0,166**	**OK**
Axial load N_{Ed}	0,0	kN	$N_{c,Rd}$	472,1		$N_{b,Rd}$			
Combined $M_{Ed,y}$ (max) + N_{Ed}		(strong)							
		(weak)							

CROSS SECTION STRENGHT AT POINT LOAD:

At point load	**M_{Ed1}**	---	kNm	$M_{c,Rd}$	---	**0**	$M_{f,Rd}$	---	**0**
	V_{Ed1}	---	kN	$V_{pl,Rd}$	---	**0**	$V_{bu,Rd}$	---	**0**
At point load	**M_{Ed2}**	---	kNm	$M_{c,Rd}$	---	**0**	$M_{f,Rd}$	---	**0**
	V_{Ed2}	---	kN	$V_{pl,Rd}$	---	**0**	$V_{bu,Rd}$	---	**0**
At point load	**M_{Ed3}**	---	kNm	$M_{c,Rd}$	---	**0**	$M_{f,Rd}$	---	**0**
	V_{Sd3}	---	kN	$V_{pl,Rd}$	---	**0**	$V_{bu,Rd}$	---	**0**

SERVICE LIMIT STATE:				DEFLECTION mm	= span /	limit span/	mm	actual/ limit	
Max moment	M_{max}	12	kNm	11,8	356	200	21,0	**0,562**	**OK**
Moment / live	M_q	11	kNm	10,7	394	250	16,8	**0,634**	**OK**

Natural frequency of uniformly loaded beam — Design criteria for natural frequency: f_1 > 3,0 Hz

f_1 = ∏/(2*L^2) * SQRT((EI)/m)) = **8,52 Hz**

m = DL + ψ_2 * LL = 199,3 kg/m — ψ_2 = 0,3 Quasi-permanent combination

Appendix 2

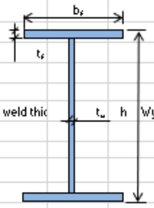
Calculation of the profile of beam in case: Beam 1-IPE360 /Beam 2-IPE160 ------Beam 1-IPE360 (example)

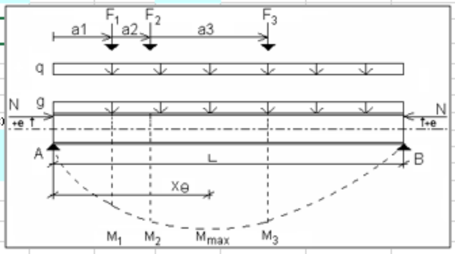
SWECO	DESIGN OF SINGLE SPAN STEEL MEMBER SYMMETRIC I -SECTION	TO EN 1993-1-1	DATE	18.2.2020

PROJECT:	Insinöörityö	ROLLED PROFILE:		WELDED BUILT UP STEEL SHAPE
AREA	Esimerkkitaso B=6.2 m, L=8.4 m	WELDED BEAM:	400-5/12x220	H-t_w/t_f xb (depth-web/t,flange x b,flange) (mm)
BEAM	BEAM1	BEAM WEIGTH	472 kg	

Steel member type: **BEAM**

It =	3,23E+05 mm4	h =	400 mm	STEEL GRADE:	**S355**	GROSS SECTION CLASS:	
		t_w =	5 mm				
Iw =	8,01E+11 mm6	t_f =	12 mm	f_y =	355 N/mm2	web:	3
Iz =	2,13E+07 mm4	b_f =	220 mm	f_u =	470 N/mm2	flange:	3
Wz =	1,936E+05 mm3	r =	0,0 mm	G =	80800 N/mm2		
i_z =	54,5 mm	A =	7160 mm2	E_s =	210000 N/mm2		
G =	56,2 kg/m	Iy =	2,209E+08 mm4	βw =	0,9 factor		
Web/flange weld =	2,50 mm (total weld thic	Wy =	1,104E+06 mm3	ε = √235/fy =	0,814		
PLASTIC PROPERTIES		i_y =	175,6 mm	βø =	0,92		
W_{pl} =	1,20E+06 mm3						
$M_{pl,Rd}$ =	**426,4** kNm						





LOADINGS ON MEMBER (nominal loads)

(to be filled in numerical order, value = 0, if load doesn't exist)

				Dead load portion (%)	Live load (%)
F1 (kN) -	0,00	a1(m)=	0,00	0	0
F2 (kN) -	0,00	a2(m)=	0,00	0	0
F3 (kN) -	0,00	a3(m)=	0,00	45	0
Fd1 (kN) -	0		γ F1 =	0	
Fd2 (kN) -	0		γ F2 =	0	
Fd3 (kN) -	0		γ F3 =	0	
Axial load N (kN) =	0,00	Dead load portion (%) -		30	

g (DL) (kN/m)=	1,612	gd =	2,49 kN/m	L (m) =	8,40	N_{Ed}(kN) =	0,0
q (LL) (kN/m)=	12,4	qd =	18,6 kN/m			Compression is positive	

Excentricity e(mm) = 0

MOMENT AND SHEAR in ultimate limet state: Load factors: γG = 1,15 γQ = 1,5 for distributed loads

Reactions — X_0 = 4,20 m

ULS R_{AGd}=	10,4 kN	Design values:	M_{max}= M_{Ed}=	**186,00** kNm		R_{BGd}=	10,4 kN
R_{AQd}=	78,1 kN		V_{Ed}=	**88,6** kN		R_{BQd}=	78,1 kN
	88,6 kN	M_{Ed1} =	--- kNm	V_{Ed1}=	--- kN		88,6 kN
SLS R_A =	61,2 kN	M_{Ed2} =	--- kNm	V_{Ed2}=	--- kN	R_B =	61,2 kN (SLS)
		M_{Ed3} =	--- kNm	V_{Ed3}=	--- kN		

DESIGN CRITERIA

	γ M2 =	1,25	BUCKLING;		DEFLECTION LIMITS:
Partial safety factors	γ M0, γ M1 =	1,00	buckling length(y)=	8,4 m	total deflectio L/ 350
Web stiffener spacing	a =	8400 mm	buckling length(z)=	8,4 m (weak)	live load L/ 400
Factor / lateral torsional buckling (EC3:6.3.2.3)	λLT,0 =	0,2	as specified in NA		
Factor / lateral torsional buckling (EC3:6.3.2.3)	β =	1	as specified in NA	Load combination in deflection check	
Shear strength/ Tensile yield limit (EC3:5.1)	η =	1,0		g_k+F+ψq_k , ψ factor for variable actions =	1,0

CALCULATION OF ELASTIC CRITICAL MOMENT IN LATERAL BUCKLING — Calculaton method for lateral bucklir General case, 6.3.2.2

k_w=1,0 ja k=1,0 (beam ends are not fixed,= fork support at beam ends) — free span / compression flange = 4,2 m

C_1 = 1,127 equivalent uniform load on beam span — z_g= 200 mm, distance from load point to the centroid of beam

C_2 = 0,454 factor for transverse loading of the beam — + when load on top flange

(C_2 =0 , if bending moment diagram is linear between lateral restraints or transverse load is applied at shear centre of the beam)

COMBINED AXIAL COMPRESSION FORCE AND BENDING:

Factor for equivalent equally distributed moment M_y : C_{my} = 0,95 depends on moment diagram shape on the overall length of beam value 0,4...1,0. see Table B.3 in EN 1993-1-1

ANALYSIS AND DESIGN SUMMARY:

ACTIONS ON STEEL MEMBER			STRENGTH OF CROSS SECTION			STABILITY OF STEEL MEMBER			NOTE ! / ADDITIONAL CHECK
Action	value	unit	Capacity	value	rate E_d/R_d	Capacity	value	rate E_d/R_d	
Moment M_{Edy}	186,0	kNm	$M_{c,Rd}$	392,0	**0,474**	$M_{yb,Rd}$	217,6	**0,855**	**OK**
Shear V_{Ed}	88,6	kN	$V_{c,Rd}$	299,2	**0,296**	$V_{c,Rd}$	299,2	**0,296**	**OK**
Axial load N_{Ed}	0,0	kN	$N_{c,Rd}$	2541,8		$N_{b,Rd}$			
Combined M_{Edy} (max) + N_{Ed}		(strong)							
		(weak)							

CROSS SECTION STRENGHT AT POINT LOAD:

At point load	**M_{Ed1}**	---	kNm	$M_{c,Rd}$	---	**0**	$M_{f,Rd}$	---	**0**
	V_{Ed1}	---	kN	$V_{pl,Rd}$	---	**0**	$V_{bu,Rd}$	---	**0**
At point load	**M_{Ed2}**	---	kNm	$M_{c,Rd}$	---	**0**	$M_{f,Rd}$	---	**0**
	V_{Ed2}	---	kN	$V_{pl,Rd}$	---	**0**	$V_{bu,Rd}$	---	**0**
At point load	**M_{Ed3}**	---	kNm	$M_{c,Rd}$	---	**0**	$M_{f,Rd}$	---	**0**
	V_{Sd3}	---	kN	$V_{pl,Rd}$	---	**0**	$V_{bu,Rd}$	---	**0**

SERVICE LIMIT STATE:				DEFLECTION mm	= span /	limit span/	mm	actual/ limit	
Max moment	M_{max}	128	kNm	20,4	413	350	24,0	**0,848**	**OK**
Moment / live	M_q	109	kNm	17,3	485	400	21,0	**0,825**	**OK**

Natural frequency of uniformly loaded beam — Design criteria for natural frequency: f_1 > 3,0 Hz

$f_1 = \pi/(2*L^2) * SQRT((EI)/m))$ = **6,19 Hz**

$m = DL + \psi_2 * LL$ = 599,7 kg/m — ψ_2 = 0,3 Quasi-permanent combination

Appendix 3

Ekvivalentin ortotrooppisen levyn materiaaliarvojen laskenta

Ekvivalentin ortotrooppisen levyn materiaaliarvojen laskenta

X-suunta

$A := 1000\text{mm}\cdot 30\text{mm} = 0.03\,\text{m}^2$ $\quad F := 1000\text{N}$

$\sigma_x := \frac{F}{A} = 0.033\cdot\text{MPa}$ $\quad \varepsilon_x := 1.7067\text{e-}006$

$E_x := \frac{\sigma_x}{\varepsilon_x} = 1.953\times 10^4\cdot\text{MPa}$

$\varepsilon_y := -6.1618\text{e-}007$

$\nu_{xy} := -\frac{\varepsilon_y}{\varepsilon_x} = 0.361$

Y-suunta

$\sigma_y := \frac{F}{A} = 0.033\cdot\text{MPa}$ $\quad \varepsilon_{y2} := 1.4035\text{e-}005$

$E_y := \frac{\sigma_y}{\varepsilon_{y2}} = 2.375\times 10^3\cdot\text{MPa}$

$\varepsilon_{x2} := -6.0294\text{e-}006$

$\nu_{yx} := -\frac{\varepsilon_{x2}}{\varepsilon_{y2}} = 0.43$

$\frac{\nu_{yx}}{E_y} = 1.809\times 10^{-10}\,\frac{1}{\text{Pa}}$ $\quad$ <==> $\quad \frac{\nu_{xy}}{E_x} = 1.849\times 10^{-11}\,\frac{1}{\text{Pa}}$

XY-suunta

$\tau_{xy} := \frac{F}{A}$ $\quad \gamma_{xy} := \frac{(0.86278 + 0.85846 + 0.90526 + 0.90524)\text{mm}}{1000\text{mm}} = 3.532\times 10^{-3}$

$G_{xy} := \frac{\tau_{xy}}{\gamma_{xy}} = 9.438\cdot\text{MPa}$

Appendix 4/1

Mathcad calculation of the effective buckling length of beam in case: Beam 1-IPE360 /Beam 2-IPE160 ------Beam 1-IPE360 (example)

Parameters

E = Young modulus
G = Shear modulus
k = Effective length factor which is related to the restrain against lateral bending at the boundaries
k_w = Effective length factor which is related to the restrain against warping at the boundaries
z_g = Distance between the point of load application and the shear centre
C_1 = Factor that account for the shape of the moment diagram
C_2 = Factor that account for the point of load application in relation to the shear centre
f_y = Yield strength

Buckling lenght calculation

Material properties

$E := 210GPa$ $G := 81GPa$ $f_y := 355MPa$

Beam length: $L_{beam} := 8.482m$

Cross section properties of the beam:

$h := 360mm$ $b_f := 170mm$ $t_f := 12.7mm$ $t_w := 8mm$ $h_w := h - 2 \cdot t_f = 334.6 \cdot mm$

$$I_y := 2 \cdot b_f \cdot t_f \cdot \left(\frac{h - t_f}{2}\right)^2 + \frac{1}{12} \cdot t_w \cdot h_w^3 = 1.552 \times 10^4 \cdot cm^4$$

$$I_z := 2 \cdot \frac{1}{12} \cdot b_f^3 \cdot t_f + \frac{1}{12} \cdot t_w^3 \cdot h_w = 1.041 \times 10^3 \cdot cm^4$$

$$I_t := 2 \cdot \frac{1}{3} \cdot t_f^3 \cdot b_f + \frac{1}{3} \cdot t_w^3 \cdot h_w = 28.926 \cdot cm^4$$

$$I_\omega := \frac{(h_w + t_f)^2 \cdot I_z}{4} = 3.14 \times 10^5 \cdot cm^6$$

$$I_\omega := \frac{1}{24} \cdot b_f^3 \cdot (h_w + t_f)^2 \cdot t_f = 3.136 \times 10^5 \cdot cm^6$$

$$W_y := \frac{I_y}{\frac{h}{2}} = 862.113 \cdot cm^3$$

Correct if needed plastic capacity.

Critical buckling moment

Maximum load $M_{Ed} := 126.57kN \cdot m$

critical load amplifier $\alpha_{cr.op} := 3.388$

lateral torsional buckling load acc. fem: $M_{cr} := \alpha_{cr.op} \cdot M_{Ed} = 428.819 \cdot kN \cdot m$

The comparison value:
- Load at top of the beam
- kω=1 and k =1
- the uniform load distribution
- the full buckling length

$C_1 := 1.127$ $C_2 := 0.454$ $L_{cr.full} := L_{beam}$ $z_g := \frac{h}{2} = 0.18m$

$$M_{cr.comp} := C_1 \cdot \frac{\pi^2 \cdot E \cdot I_z}{L_{cr.full}^2} \cdot \left[\sqrt{\frac{I_\omega}{I_z} + \frac{L_{cr.full}^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z} + (C_2 \cdot z_g)^2} - C_2 \cdot z_g\right] = 86.971 \cdot kN \cdot m$$

$$\frac{M_{cr}}{M_{cr.comp}} = 4.931$$

Moment resistance:

Buckling curve	a	b	c	d
Imperfection factor α_{LT}	0,21	0,34	0,49	0,76

$\alpha_{LT} := 0.34$ Imperfection factors

$$\lambda_{LT} := \sqrt{\frac{W_y \cdot f_y}{M_{cr}}} = 0.845 \qquad \lambda_{LT.comp} := \sqrt{\frac{W_y \cdot f_y}{M_{cr.comp}}} = 1.876$$

$$\phi_{LT} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{LT} - 0.2) + \lambda_{LT}^2\right] = 0.966 \qquad \phi_{LT.comp} := 0.5 \cdot \left[1 + \alpha_{LT} \cdot (\lambda_{LT.comp} - 0.2) + \lambda_{LT.comp}^2\right] = 2.544$$

$$\chi_{LT} := \min\left(1, \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \lambda_{LT}^2}}\right) = 0.696 \qquad \chi_{LT.comp} := \min\left(1, \frac{1}{\phi_{LT.comp} + \sqrt{\phi_{LT.comp}^2 - \lambda_{LT.comp}^2}}\right) = 0.235$$

$$M_{Rd} := \chi_{LT} \cdot W_y \cdot f_y \qquad M_{Rd.comp} := \chi_{LT.comp} \cdot W_y \cdot f_y$$

$$\frac{M_{Rd}}{M_{Rd.comp}} = 2.969$$

NOTE: The critical buckling moment Mcr is needed for the calculation of moment resistance - not the LTB lenght

LTB buckling length

a) The buckling length

The load is at centerline of the beam:

$$M_{cr} = \frac{\pi^2 \cdot E \cdot I_z}{L_{cr}^2} \cdot \sqrt{\frac{I_\omega}{I_z} + \frac{L_{cr}^2 \cdot G \cdot I_t}{\pi^2 \cdot E \cdot I_z}} \text{ solve}, L_{cr} \rightarrow$$

Appendix 5

Results and sensitivity tests of narrow-flange primary I-beam for both cases of factor C_1=1 and C_1=1.127

secondary beam IPE140

C_1=1	L(m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)	C_1=1.127	L(m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)
IPE200	6.073	1.841	0.30	IPE200	6.073	1.981	0.33
IPE270	8.473	2.570	0.30	IPE270	8.473	2.760	0.33
IPE360	10.873	3.557	0.33	IPE360	10.873	3.821	0.35

secondary beam IPE160

C_1=1	L (m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)	C_1=1.127	L (m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)
IPE240	6.082	2.000	0.33	IPE240	6.082	2.145	0.35
IPE360	8.482	3.196	0.38	IPE360	8.482	3.427	0.40
WI400-5-15*220	10.882	5.295	0.49	WI400-5-15*220	10.882	5.700	0.52

secondary beam IPE220

C_1=1	L (m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)	C_1=1.127	L (m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)
WI400-5-12*220	8.510	4.669	0.55	WI400-5-12*220	8.510	4.996	0.59
WI450-5-16*250	10.910	6.027	0.55	WI450-5-16*250	10.910	6.480	0.59

secondary beam IPE220 (Sensitivity Test)

TEST (C_1=1)	L (m)	L_{cr} (m)	FACTOR ((k_{LT} =L_{cr}/L)	TEST (C_1=1.127)	L (m)	L_{cr} (m)	FACTOR (k_{LT} =L_{cr}/L)
WI400-5-12*220	8.510	4.909	0.58	WI400-5-12*220	8.510	5.257	0.62
WI450-5-16*250	10.910	6.231	0.57	WI450-5-16*250	10.910	6.705	0.61

Appendix 6

Mathcad calculation of how much capacity of reduction factor increases in case: Beam 1- WI450-5-16*250 /Beam 2-IPE140 (example)

Material properties

$E := 210GPa$ $G := 81GPa$ $f_y := 355MPa$

Beam length: $L_{beam} := 10.91m$

Cross section properties of the beam:

$h := 450mm$ $b_f := 250mm$ $t_f := 16mm$ $t_w := 5mm$ $h_w := h - 2\cdot t_f = 418\cdot mm$

$$I_y := 2\cdot b_f\cdot t_f\cdot\left(\frac{h-t_f}{2}\right)^2 + \frac{1}{12}\cdot t_w\cdot h_w^3 = 4.071\times 10^4\cdot cm^4$$

$$I_z := 2\cdot\frac{1}{12}\cdot b_f^3\cdot t_f + \frac{1}{12}\cdot t_w^3\cdot h_w = 4.167\times 10^3\cdot cm^4$$

$$I_t := 2\cdot\frac{1}{3}\cdot t_f^3\cdot b_f + \frac{1}{3}\cdot t_w^3\cdot h_w = 70.008\cdot cm^4$$

$$I_\omega := \frac{(h_w + t_f)^2\cdot I_z}{4} = 1.962\times 10^6\cdot cm^6$$

$$I_\omega := \frac{1}{24}\cdot b_f^3\cdot(h_w + t_f)^2\cdot t_f = 1.962\times 10^6\cdot cm^6$$

$$W_y := \frac{I_y}{\frac{h}{2}} = 1.81\times 10^3\cdot cm^3$$

Correct if needed plastic capacity.

$\alpha_{LT} := 0.49$ Imperfection factors

RFEM results:

$M_{cr.FEM} := 633.31kN\cdot m$

$$\lambda_{LT.FEM} := \sqrt{\frac{W_y\cdot f_y}{M_{cr.FEM}}} = 1.007$$

The comparison value:
- Load at top of the beam
- kω=1 and k =1
- the uniform load distribution
- the full buckling length

$C_1 := 1.127$ $C_2 := 0.454$ $L_{cr.full} := L_{beam}$ $z_g := \frac{h}{2} = 0.225\,m$

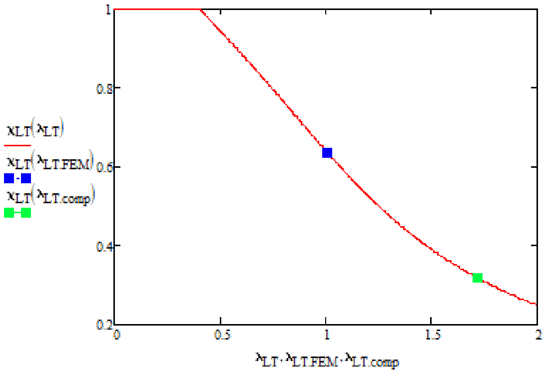
$\beta := 0.75$

$$M_{cr.comp} := C_1\cdot\frac{\pi^2\cdot E\cdot I_z}{L_{cr.full}^2}\cdot\left[\sqrt{\frac{I_\omega}{I_z} + \frac{L_{cr.full}^2\cdot G\cdot I_t}{\pi^2\cdot E\cdot I_z} + (C_2\cdot z_g)^2} - C_2\cdot z_g\right] = 217.675\cdot kN\cdot m$$

$$\lambda_{LT.comp} := \sqrt{\frac{W_y\cdot f_y}{M_{cr.comp}}} = 1.718$$

$$\phi_{LT}(\lambda_{LT}) := 0.5\cdot\left[1 + \alpha_{LT}\cdot(\lambda_{LT} - 0.4) + \beta\cdot\lambda_{LT}^2\right]$$

$$\chi_{LT}(\lambda_{LT}) := \min\left[1, \frac{1}{\phi_{LT}(\lambda_{LT}) + \sqrt{\phi_{LT}(\lambda_{LT})^2 - \beta\cdot(\lambda_{LT})^2}}\right]$$



$$\frac{\chi_{LT}(\lambda_{LT.FEM})}{\chi_{LT}(\lambda_{LT.comp})} = 2.005$$