Preliminary engineering calculations and design assumptions for Cox's walk footbridge alternative repair proposal, July 2020_RevB

Design standards / codes used in the structural calculations;

- DMRB CD 353 – Design Criteria for footbridges, Mar 2020
- BS EN 1990, Eurocode 0: Basis of structural design
- BS EN 1991-2, Eurocode 1: Actions on structures. Traffic loads on bridges
- BS EN 1992-1-1_2004, Eurocode 2: Design of concrete structures General rules and rules for buildings
- BS EN 1993-1-1_2005, Eurocode 3: Design of steel structures General rules and rules for buildings
- BS EN 1995-1-1_2004, Eurocode 5: Design of timber structures Common rules and rules for buildings
- BS EN 1996-1-1_2005, Eurocode 6: Design of masonry structures General rules for reinforced and unreinforced masonry structures

Introduction;

This is a preliminary calculation report for the alternative repair proposal for the Cox's walk footbridge. This report includes calculations for the splice connection proposed to extend the approx. 9.1m span 305UC to a 10.8m span 305UC, and an associated check of the natural frequency of the beam and a historic steelwork stress check of the beam. A calculation is presented justifying the 80mm THK RC slab required to eliminate any increase in dead load on the central (West) pier, including a check under a 10kN point load as per BS EN 1991-2_2003.

Tekla Structural Designer 2019i has been used for the structural analysis and to determine the approx. max. screw pile loads (62kN) which is given in unfactored (SLS) format in the notes on drawing 001, which is believed to be within the capacity of a screw pile installed in such circumstances although this would need to be confirmed with a piling contractor in relation to site investigation. The SLS pile loads could be further refined at the following stage of design.

For the purposes of discussions with screw pile contractors, if the contractor's design is governed by settlement it should be noted that the 62kN SLS load given includes the full 5kN/m^2 imposed load without any 'quasi-permanent' load reduction which would typically be associated with assessing serviceability in relation to long-term settlement.

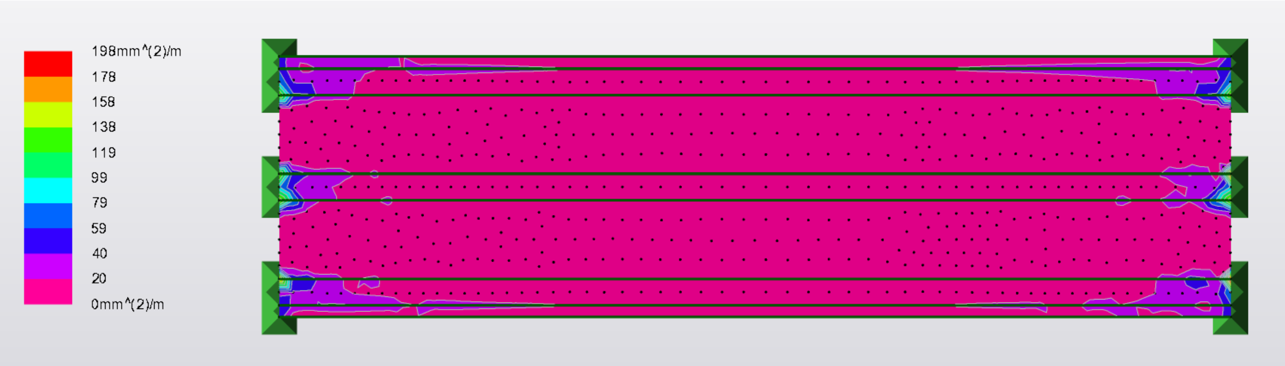
Live Loading;

The design live load has been determined as per BS EN 1991-2_2003, Clause 5.3.2.1 – Uniformly distributed load. A live load of 5.0 kN/m^2 (considering continuous, dense crowding) has been used in the analysis and design. This could be reduced to 4.0 kN/m^2 (in accordance with 5.3.2.1 (2) (5.1) at the client's request if continuous, dense crowding is not required due to the nature and setting of this footbridge.

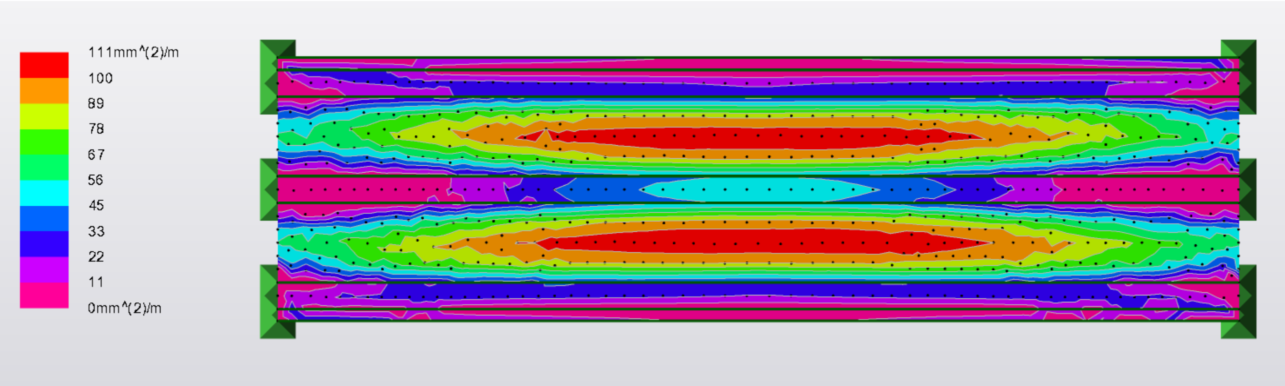
A new Reinforced Concrete Deck Slab of 80mm thickness is proposed over the new 10.8m span 305x137 UC beams to remove any increase in dead load on the left central masonry pier due. The analysis below has been carried out taking into account the stiffness of the steel beams and is for the load combination (1.35Gk + 1.5Qk). It can be seen that the slab generally requires very little reinforcement due to the very small spans of the slab (900mm between flanges of steel beam), and so nominal reinforcement of A193 mesh is adequate. The point load requirement of BS EN 1991-2 has also been met.

Reinforcement required in 80mm THK RC slab (35mm cover top – 30mm cover bottom) RC32/40 concrete, fixing tolerance 5mm.

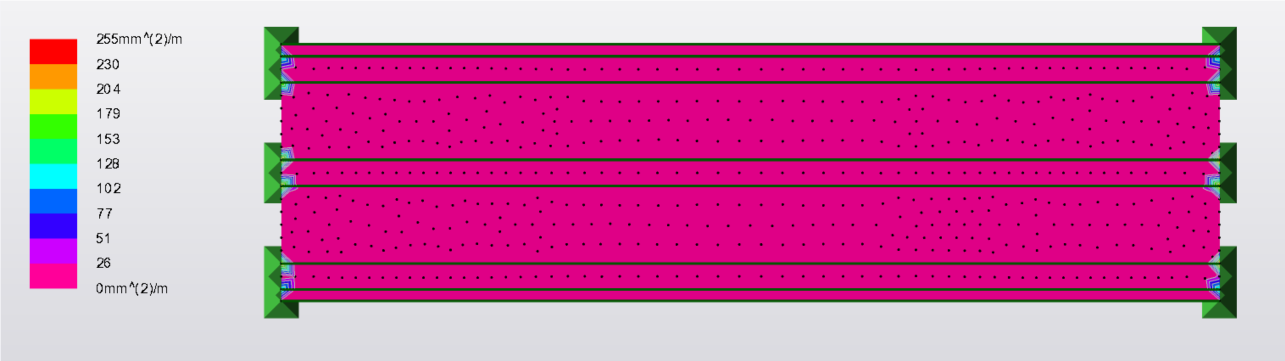
X top



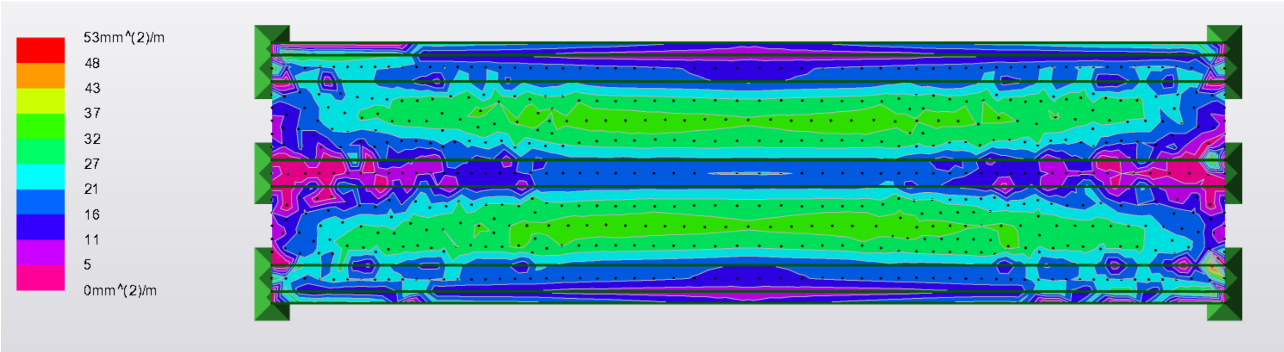
X bottom



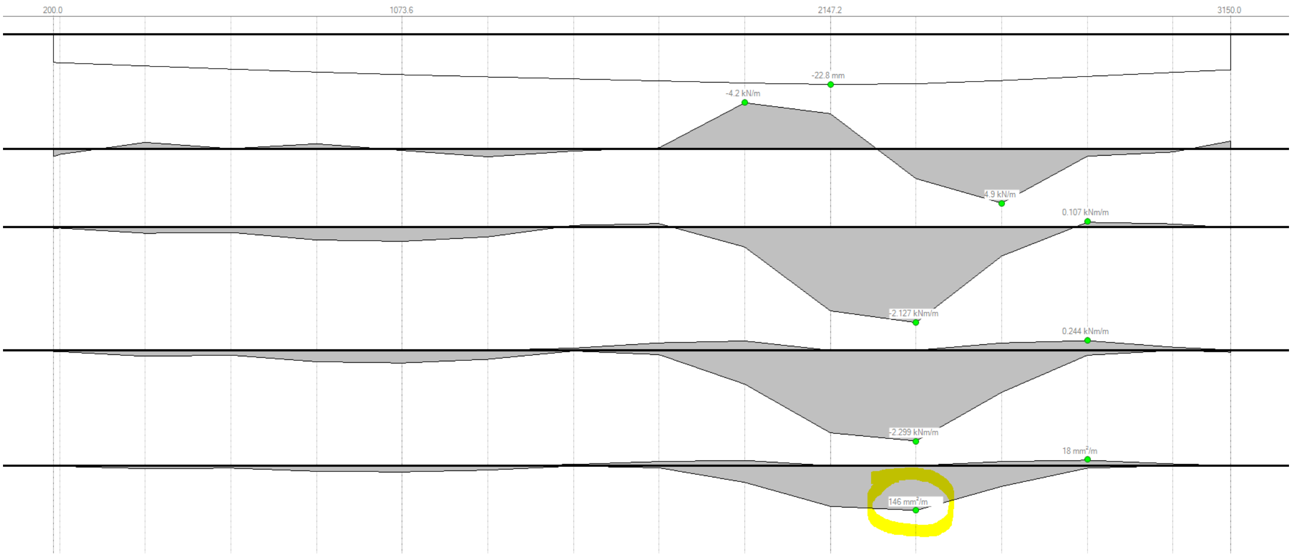
Y top

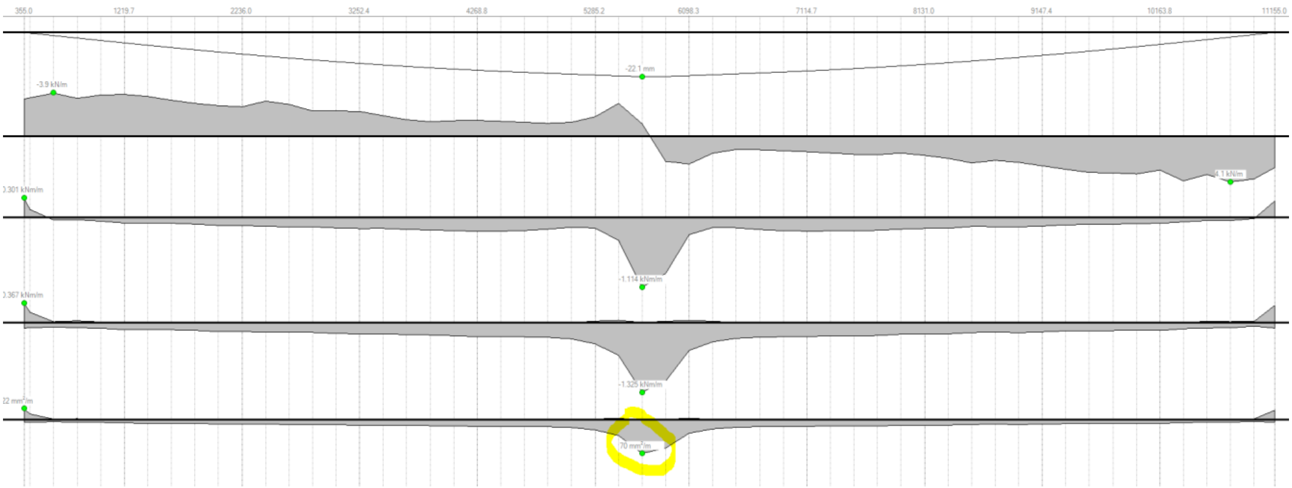


Y bottom



10kN Point Load in accordance with NA to BS EN 1991-2:2003;



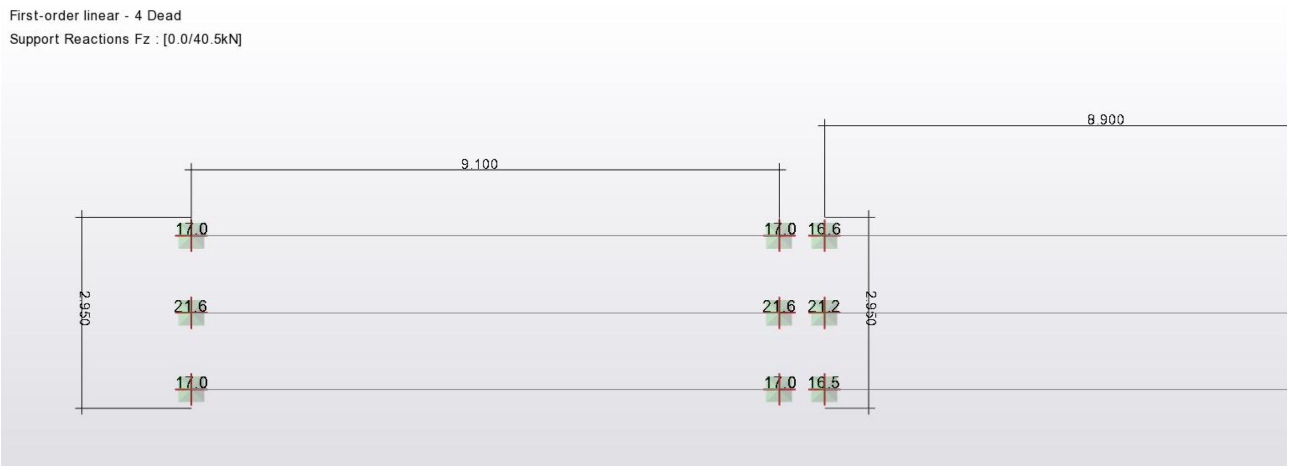


A193 or B196 mesh fine by inspection, 5mm bars at 100C-C laid parallel to the steel beams, 7mm bars at 200C-C laid across the steel beams.

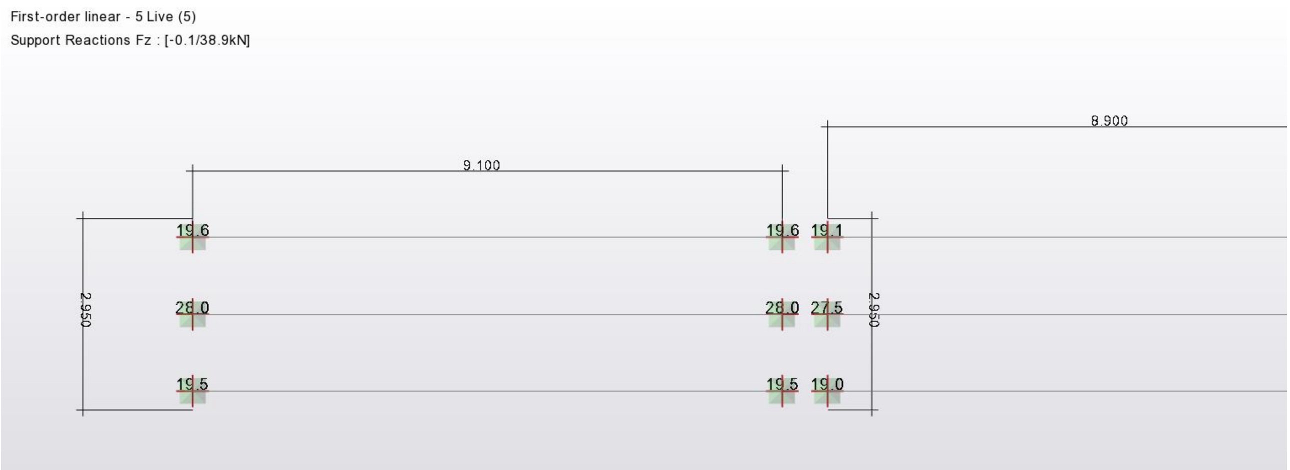
Increase in load calculation on Central (West) Pier due to extending beam

Existing Reactions on left pier from existing bridge

Dead load

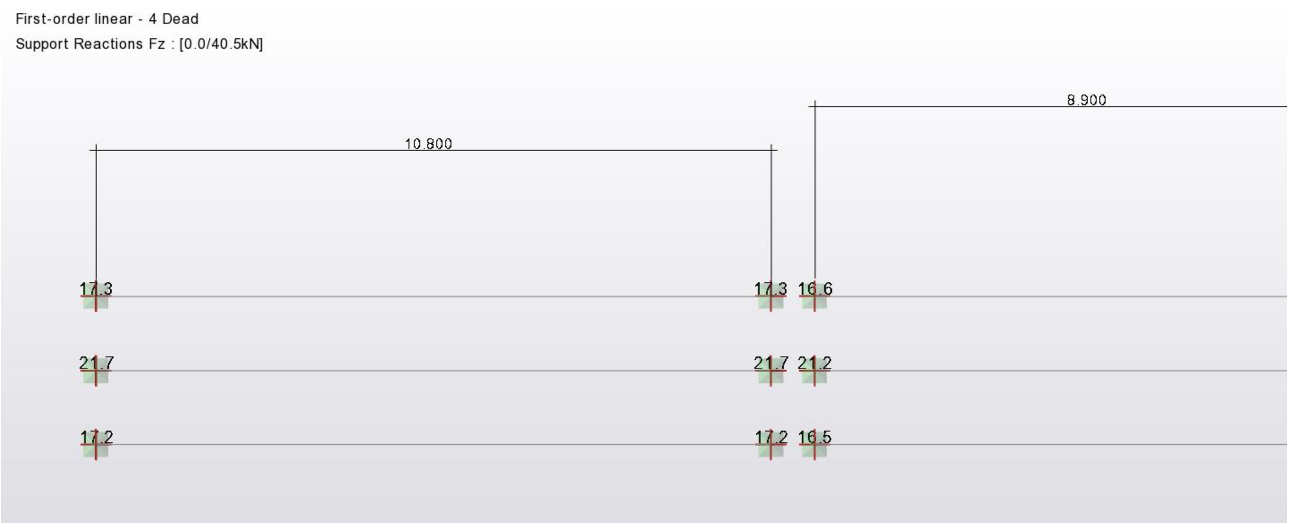


Live load



Proposed Reactions on left pier from proposed bridge (load-sharing with truss not assumed)

Dead load



Live load

Change in load on left pier;

Dead load

Proposed Load / Existing Load

= 17+21.6+17+16.6+21.2+16.5 / 17+21.6+17+16.6+21.2+16.5 = 110kN / 110kN = no change in load

Live load

= 23.2+19.1+33.5+27.5+23+19 / 19.6+19.1+28+27.5+19.5+19 = 145kN / 133kN = 12kN or 9% increase in load (< 10%).

Proposed existing steel beam (10.8m span) design check (1950s steelwork allowable stress approach – lateral restraint provided by deck and tie beams)

Allowable stress = 230 / 1.3 = 177 N/mm^2

ULS Bending Moment = 185kNm

<u>305x305x118 UC;</u>

Zyy = 1760 x 10^3 Nmm^3

Stress = 185 * 10^6 / 1760 x 10^3 = 105 N/mm^2

Utilisation = 105 / 177 = 0.59

<u>305x305x97 UC</u> Stress = 185 * 10^6 / 1450 x 10^3 = 128 N/mm^2

Utilisation = 128 / 177 = 0.72

Beam deflection check <u>deflection = 23.2mm = L/470 < L/250 therefore okay;</u>

SB Base/%313-Base/%316 results (BS EN 1993-1-1 + UK NA, 2005)

Summary UC 305x305x137(S235)
- Classification
- Shear Major
- Shear Minor
- Buckling Shear Web
- Moment Major
- Moment Minor
- Axial
- Axial Bending Combined
- Buckling Lateral Torsional
- Buckling Compression
- Buckling Combined
- Torsion
- Deflection

Summary UC 305x305x137(S235)

Design Condition	#	Design Value	Design Capacity	Units	U.R.	Status
Classification	1	Class 1	-	-	-	Pass
Shear Major	1	71.9	647.1	kN	0.111	Pass
Shear Minor	-	No	Forces	kN	-	Not required
Buckling Shear Web	-	20.080	73.583	-	-	Pass
Moment Major	1	184.9	516.7	kNm	0.358	Pass
Moment Minor	-	No	Forces	kNm	-	Not required
Axial	-	No	Forces	kN	-	Not required
Axial Bending Combined	-	No	Forces	-	-	Not required
Buckling Lateral Torsional	1	184.9	428.2	kNm	0.432	Pass
Buckling Compression	-	No	Forces	-	-	Not required
Buckling Combined	-	No	Forces	-	-	Not required
Torsion	-	No	Significant	Forces	-	Not required
Deflection Self weight	1	3.5	-	mm	-	-
Deflection Slab	1	5.4	43.2	mm	0.125	Pass
Deflection Dead	1	0.5	21.6	mm	0.025	Pass
Deflection Imposed	1	13.7	30.0	mm	0.458	Pass
Deflection Total	1	23.2	43.2	mm	0.536	Pass

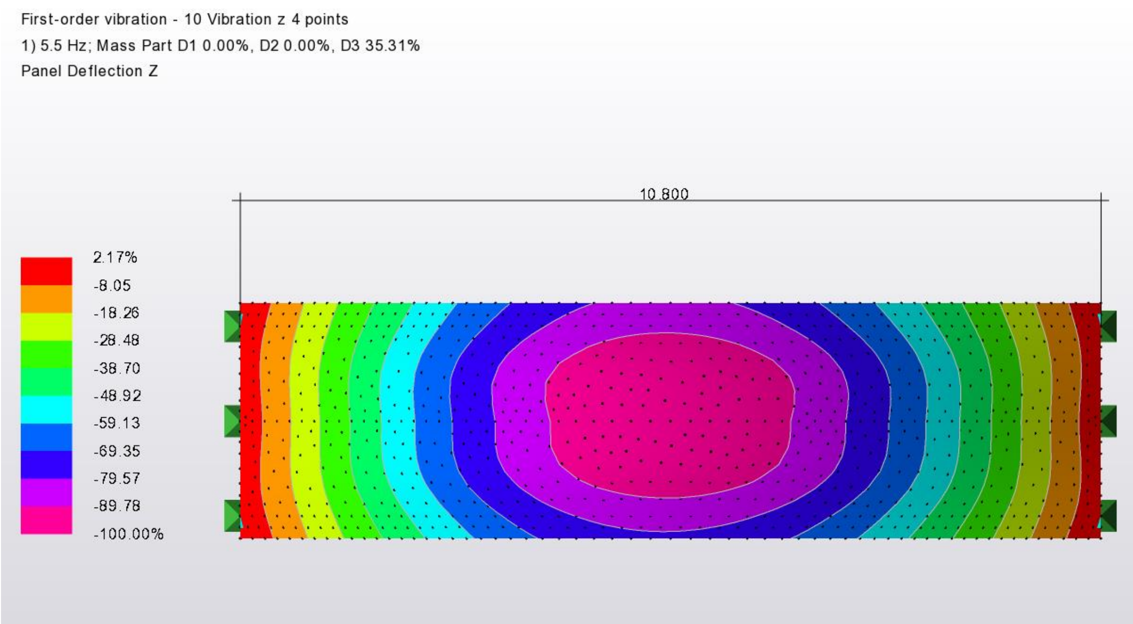
Settings   Expand All   Collapse All   Close

Natural Frequency check

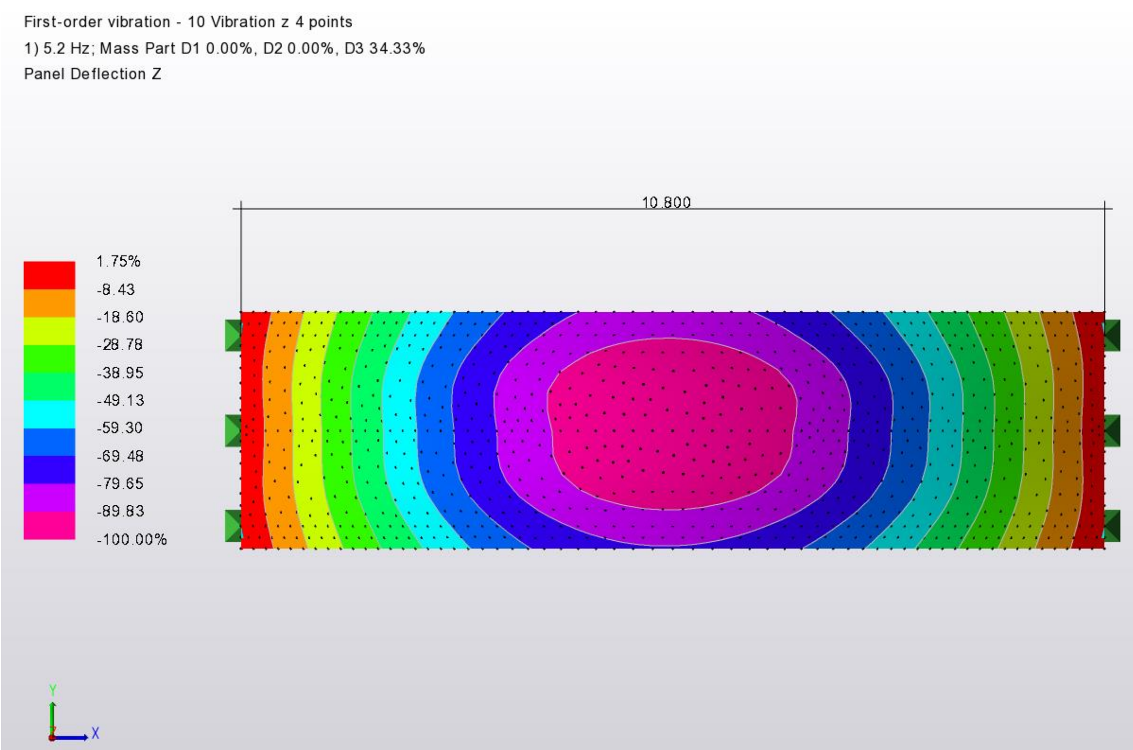
Vibration analysis has been carried out for 2 beam sizes, 305x305x137 UC which it is suspected is the actual size of beam, and 305x305x118 UC to account for effects of a nominal amount of corrosion.

The load case used considers 100% dead load and 100% live load considering 4 No. 100kg people running across the bridge closely spaced and at mid-span of the bridge.

305x305x137 UC - Frequency of 10.8m span = 5.5Hz > 3Hz therefore okay;

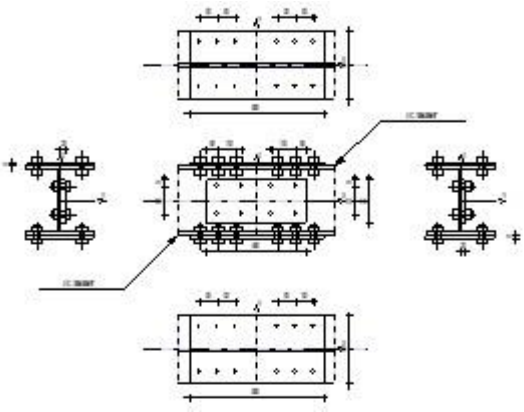


305x305x118 UC - Frequency of 10.8m span = 5.2Hz > 3Hz therefore okay;



Proposed splice detail; (305x305x97 UC assumed as beam showing minor corrosion and to allow for long lifespan of detail – 6.3mm section loss of flange and 3.9mm section loss of web – 0.3 Utilisation)

	Autodesk Robot Structural Analysis Professional 2019 **Calculation of the beam-to-beam splice connection** EN 1993-1-8:2005/AC:2009	OK Ratio **0.30**



GENERAL

Connection no.: 6

Connection name: West 305x305x138 Splice

RIGHT BEAM

Section: UC 305x305x97

h_{b1} =	308	[mm]	Height of beam section
b_{fb1} =	305	[mm]	Width of beam section
t_{wb1} =	10	[mm]	Thickness of the web of beam section
t_{fb1} =	15	[mm]	Thickness of the flange of beam section
r_{b1} =	15	[mm]	Radius of beam section fillet
A_{b1} =	12300	[mm^2]	Cross-sectional area of a beam
I_{yb1} =	222500000	[mm^4]	Moment of inertia of the beam section

Material: S275

f_{yb1} =	275.00	[MPa]	Resistance
f_{ub1} =	430.00	[MPa]	

LEFT BEAM

Section: UC 305x305x97

h_{b2} =	308	[mm]	Height of beam section
b_{fb2} =	305	[mm]	Width of beam section
t_{wb2} =	10	[mm]	Thickness of the web of beam section
t_{fb2} =	15	[mm]	Thickness of the flange of beam section
r_{b2} =	15	[mm]	Radius of beam section fillet
A_{b2} =	12300	[mm^2]	Cross-sectional area of a beam
I_{yb2} =	222500000	[mm^4]	Moment of inertia of the beam section

Material: S275

f_{yb2} =	275.00	[MPa]	Resistance
f_{ub2} =	430.00	[MPa]	

SPLICE PLATE

Type: unilateral

l_{pw} =	450	[mm]	Plate length
h_{pw} =	200	[mm]	Plate height
t_{pw} =	20	[mm]	Plate thickness
Material:	S355		
f_{ypw} =	355.00	[MPa]	Design resistance
f_{upw} =	490.00	[MPa]	Tensile resistance

UPPER EXTERNAL PLATE

l_{pe} =	600	[mm]	Plate length
h_{pe} =	308	[mm]	Plate height
t_{pe} =	15	[mm]	Plate thickness
Material:	S355		
f_{ype} =	355.00	[MPa]	Design resistance
f_{upe} =	490.00	[MPa]	Tensile resistance

LOWER EXTERNAL PLATE

l_{pe} =	600	[mm]	Plate length
h_{pe} =	308	[mm]	Plate height
t_{pe} =	15	[mm]	Plate thickness
Material:	S355		
f_{ype} =	355.00	[MPa]	Design resistance
f_{upe} =	490.00	[MPa]	Tensile resistance

RIGHT SIDE

BOLTS CONNECTING A SPLICE PLATE WITH THE BEAM WEB

The shear plane passes through the UNTHREADED portion of the bolt.

Connection category A

Class =	8.8		Bolt class
d =	24	[mm]	Bolt diameter
d_0 =	26	[mm]	Bolt opening diameter
A_s =	353	[mm^2]	Effective section area of a bolt
A_v =	452	[mm^2]	Area of bolt section
f_{yb} =	640.00	[MPa]	Yield strength of bolt
f_{ub} =	800.00	[MPa]	Bolt tensile resistance
n_h =	2		Number of bolt columns
n_v =	2		Number of bolt rows

The shear plane passes through the UNTHREADED portion of the bolt.

e_1 =	38	[mm]	Level of first bolt
p_2 =	110	[mm]	Horizontal spacing
p_1 =	125	[mm]	Vertical spacing

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM TOP FLANGE

The shear plane passes through the UNTHREADED portion of the bolt.

Connection category A

Class =	8.8		Bolt class
d =	24	[mm]	Bolt diameter
d_0 =	26	[mm]	Bolt opening diameter
A_s =	353	[mm^2]	Effective section area of a bolt
A_v =	452	[mm^2]	Area of bolt section
f_{yb} =	640.00	[MPa]	Yield strength of bolt
f_{ub} =	800.00	[MPa]	Bolt tensile resistance
n_h =	1		Number of bolt columns
n_v =	3		Number of bolt rows
e_1 =	90	[mm]	Level of first bolt
p_1 =	80	[mm]	Vertical spacing

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM BOTTOM FLANGE

The shear plane passes through the UNTHREADED portion of the bolt.

Connection category A

Class =	8.8		Bolt class
d =	24	[mm]	Bolt diameter
d_0 =	26	[mm]	Bolt opening diameter
A_s =	353	[mm^2]	Effective section area of a bolt
A_v =	452	[mm^2]	Area of bolt section
f_{yb} =	640.00	[MPa]	Yield strength of bolt
f_{ub} =	800.00	[MPa]	Bolt tensile resistance
n_h =	1		Number of bolt columns
n_v =	3		Number of bolt rows
e_1 =	90	[mm]	Level of first bolt
p_1 =	80	[mm]	Vertical spacing

LEFT SIDE

BOLTS CONNECTING A SPLICE PLATE WITH THE BEAM WEB

The shear plane passes through the UNTHREADED portion of the bolt.

Connection category A

Class =	8.8		Bolt class
d =	24	[mm]	Bolt diameter
d_0 =	26	[mm]	Bolt opening diameter
A_s =	353	[mm^2]	Effective section area of a bolt
A_v =	452	[mm^2]	Area of bolt section
f_{yb} =	640.00	[MPa]	Yield strength of bolt
f_{ub} =	800.00	[MPa]	Bolt tensile resistance
n_h =	2		Number of bolt columns
n_v =	2		Number of bolt rows
e_1 =	38	[mm]	Level of first bolt
p_2 =	110	[mm]	Horizontal spacing
p_1 =	125	[mm]	Vertical spacing

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM TOP FLANGE

The shear plane passes through the UNTHREADED portion of the bolt.

Connection category A

Class =	8.8		Bolt class
d =	24	[mm]	Bolt diameter
d_0 =	26	[mm]	Bolt opening diameter
A_s =	353	[mm^2]	Effective section area of a bolt
A_v =	452	[mm^2]	Area of bolt section
f_{yb} =	640.00	[MPa]	Yield strength of bolt
f_{ub} =	800.00	[MPa]	Bolt tensile resistance
n_h =	1		Number of bolt columns
n_v =	3		Number of bolt rows
e_1 =	90	[mm]	Level of first bolt
p_1 =	80	[mm]	Vertical spacing

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM BOTTOM FLANGE

The shear plane passes through the UNTHREADED portion of the bolt.

Connection category A

Class =	8.8		Bolt class
d =	24	[mm]	Bolt diameter
d_0 =	26	[mm]	Bolt opening diameter
A_s =	353	[mm^2]	Effective section area of a bolt
A_v =	452	[mm^2]	Area of bolt section

The shear plane passes through the UNTHREADED portion of the bolt.

f_{yb} =	640.00	[MPa]	Yield strength of bolt
f_{ub} =	800.00	[MPa]	Bolt tensile resistance
n_h =	1		Number of bolt columns
n_v =	3		Number of bolt rows
e_1 =	90	[mm]	Level of first bolt
p_1 =	80	[mm]	Vertical spacing

MATERIAL FACTORS

γ_{M0} =	1.00	Partial safety factor	[2.2]
γ_{M2} =	1.25	Partial safety factor	[2.2]

LOADS

Case: Manual calculations.

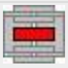
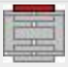
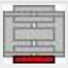
ULTIMATE LIMIT STATE

N_{Ed1} =	5.00	[kN]	Axial force
$V_{z,Ed1}$ =	47.00	[kN]	Shear force
$M_{y,Ed1}$ =	100.00	[kN*m]	Bending moment
N_{Ed2} =	0.00	[kN]	Axial force
$V_{z,Ed2}$ =	0.00	[kN]	Shear force
$M_{y,Ed2}$ =	0.00	[kN*m]	Bending moment

RESULTS

RIGHT SIDE

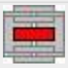
Axial force

Plate	A_i [mm2]	EQUIVALENT FORCES N_i [kN]	EQUIVALENT FORCES $N_i(M_{y,Ed})$ [kN]	Resultant force $N_{Ed,i}$ [kN]
	A_{pw}= 4000	1.51	-	**$N_{Ed,pw}$= 1.51**
	A_{pfue}= 4620	1.74	293.46	**$N_{Ed,pfue}$= 295.20**
	A_{pfle}= 4620	1.74	-293.46	**$N_{Ed,pfle}$= -291.71**

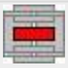
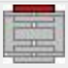
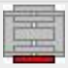
$N_i=(N_{Ed}*A_i)/(A_{wp}+A_{pfue}+A_{pfle})$

$N_{Ed,i} = N_i+N_i(M_{y,Ed})$

Shear force Z

Plate	A_i [mm2]	$V_{zEd,i}$ [kN]
	$A_{z,pw}$= 4000	$V_{z,Ed,pw}$= 47.00

Bending moment Y

Plate	$I_{y,i}$ [mm4]	EQUIVALENT FORCES $M_{y,i}$ [kN*m]	Resultant force $M_{y,Ed,i}$ [kN*m]
	$I_{y,pw}$= 13333333	5.24	**$M_{y,Ed,pw}$= 5.24**
	$I_{y,pfue}$= 120512019	47.38	-
	$I_{y,pfle}$= 120512019	47.38	-

$M_{y,i}=(M_{y,Ed}*I_{y,i})/(I_{pw}+I_{pfue}+I_{pfle})$

BOLTS CONNECTING A SPLICE PLATE WITH THE BEAM WEB

BOLT CAPACITIES

$F_{v,Rd}$ =	173.72	[kN]	Shear bolt resistance in the unthreaded portion of a bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the beam

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	$k_{1x} = \min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$		2.50 > 0.00	verified	
α_{bx} =	0.80		Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bx}=\min[e_2/(3*d_0), p_2/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$		0.80 > 0.00	verified	
$F_{b,Rd1x}$ =	163.73	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*\sum t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$	$k_{1z}=\min[2.8*(e_2/d_0)-1.7, 1.4*(p_2/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$		2.50 > 0.00	verified	
α_{bz} =	1.00		Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bz}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bz} > 0.0$		1.00 > 0.00	verified	
$F_{b,Rd1z}$ =	204.34	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}=k_{1z}*\alpha_{bz}*f_u*d*\sum t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} =	2.34		Coefficient for calculation of $F_{b,Rd}$	$k_{1x}=\min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$
$k_{1x} > 0.0$		2.34 > 0.00	verified	
α_{bx} =	0.64		Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bx}=\min[e_2/(3*d_0), p_2/(3*d_0)-0.25, f_{ub}/f_u, 1]$
$\alpha_{bx} > 0.0$		0.64 > 0.00	verified	
$F_{b,Rd2x}$ =	282.05	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2x}=k_{1x}*\alpha_{bx}*f_u*d*\sum t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$	$k_{1z}=\min[2.8*(e_2/d_0)-1.7, 1.4*(p_2/d_0)-1.7, 2.5]$
$k_{1z} > 0.0$		2.50 > 0.00	verified	

α_{bz} =	0.48	Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bz} > 0.0	0.48 > 0.00	verified	
$F_{b,Rd2z}$ =	226.15 [kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*∑t_i/γ_{M2}

ULTIMATE LIMIT STATE

Bolt shear

e_0 =	120	[mm]	Shear force eccentricity relative to the center of gravity of a bolt group	e_0 = e_{2b}+0.5*(s_1+(c-1)*p_2)
M_y =	10.88	[kN*m]	Real bending moment	M_y=$M_{y,Ed,pw}$+$V_{z,Ed,pw}$*e_0
$F_{x,N}$ =	0.38	[kN]	Component force in a bolt due to influence of the longitudinal force on the x direction	$F_{x,N}$=|$N_{Ed,pw}$|/n_b
$F_{z,Vz}$ =	11.75	[kN]	Component force in a bolt due to influence of the shear force Vz on the z direction	$F_{z,Vz}$=|$V_{z,Ed,pw}$|/n_b
$F_{x,My}$ =	24.53	[kN]	Component force in a bolt due to influence of the moment My on the x direction	$F_{x,My}$=|M_y|*z_i/∑(x_i^2+z_i^2)
$F_{z,My}$ =	21.59	[kN]	Component force in a bolt due to influence of the moment My on the z direction	$F_{z,My}$=|M_y|*x_i/∑(x_i^2+z_i^2)
$F_{x,Ed}$ =	24.91	[kN]	Design total force in a bolt on the direction x	$F_{x,Ed}$ = $F_{x,N}$+$F_{x,My}$
$F_{z,Ed}$ =	33.34	[kN]	Design total force in a bolt on the direction z	$F_{z,Ed}$ = $F_{z,Vz}$+$F_{z,My}$
F_{Ed} =	41.62	[kN]	Resultant shear force in a bolt	F_{Ed} = √($F_{x,Ed}^2$ + $F_{z,Ed}^2$)
$F_{Rd,x}$ =	163.73	[kN]	Effective design capacity of a bolt on the direction x	F_{Rdx}=min($F_{bRd1,x}$, $F_{bRd2,x}$)
$F_{Rd,z}$ =	204.34	[kN]	Effective design capacity of a bolt on the direction z	F_{Rdz}=min($F_{bRd1,z}$, $F_{bRd2,z}$)

|$F_{x,Ed}$| ≤ $F_{Rd,x}$	|24.91| < 163.73	verified	(0.15)
|$F_{z,Ed}$| ≤ $F_{Rd,z}$	|33.34| < 204.34	verified	(0.16)
F_{Ed} ≤ $F_{v,Rd}$	41.62 < 173.72	verified	(0.24)

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM TOP FLANGE

BOLT CAPACITIES

$F_{v,Rd}$ =	173.72 [kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam flange

Direction x

k_{1x} =	2.50	Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0	2.50 > 0.00	verified	
α_{bx} =	0.78	Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0	0.78 > 0.00	verified	
$F_{b,Rd1x}$ =	246.54 [kN]	Bearing resistance of a single bolt	$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1y} = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1y} > 0.0			2.50 > 0.00 verified	
α_{by} =	0.68		Coefficient for calculation of $F_{b,Rd}$	α_{by}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{by} > 0.0			0.68 > 0.00 verified	
$F_{b,Rd1y}$ =	214.55	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1y}$=k_{1y}*α_{by}*f_u*d*∑t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00 verified	
α_{bx} =	0.61		Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0			0.61 > 0.00 verified	
$F_{b,Rd2x}$ =	214.85	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1y}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1y} > 0.0			2.50 > 0.00 verified	
α_{by} =	0.69		Coefficient for calculation of $F_{b,Rd}$	α_{by}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{by} > 0.0			0.69 > 0.00 verified	
$F_{b,Rd2y}$ =	244.25	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2y}$=k_{1y}*α_{by}*f_u*d*∑t_i/γ_{M2}

ULTIMATE LIMIT STATE

Bolt shear

F_{Ed} =	51.58	[kN]	Shear force in a bolt	F_{Ed}=$N_{Ed,pfue}/n_b$
β_{Lf} =	1.00		Reduction factor for long connections	β_{Lf} = max(0.75, min(1; 1-(L-15*d)/(200*d)))
F_{Rd} =	173.72	[kN]	Effective design capacity of a bolt	F_{Rd} = min($F_{v,Rd}$; $F_{b,Rd1}$; $F_{b,Rd2}$)
$\lvert F_{Ed}\rvert \le \beta_{Lf}*F_{Rd}$			|51.58| < 173.72 verified	(0.30)

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM BOTTOM FLANGE

BOLT CAPACITIES

$F_{v,Rd}$ =	173.72	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam flange

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00 verified	
α_{bx} =	0.78		Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0			0.78 > 0.00 verified	
$F_{b,Rd1x}$ =	246.54	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} = 2.50 Coefficient for calculation of $F_{b,Rd}$ $k_{1y} = \min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$

$k_{1y} > 0.0$ 2.50 > 0.00 verified

α_{by} = 0.68 Coefficient for calculation of $F_{b,Rd}$ $\alpha_{by}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$

$\alpha_{by} > 0.0$ 0.68 > 0.00 verified

$F_{b,Rd1y}$ = 214.55 [kN] Bearing resistance of a single bolt $F_{b,Rd1y}=k_{1y}*\alpha_{by}*f_u*d*\sum t_i/\gamma_{M2}$

Bolt bearing on the plate

Direction x

k_{1x} = 2.50 Coefficient for calculation of $F_{b,Rd}$ $k_{1x}=\min[2.8*(e_2/d_0)-1.7, 2.5]$

$k_{1x} > 0.0$ 2.50 > 0.00 verified

α_{bx} = 0.61 Coefficient for calculation of $F_{b,Rd}$ $\alpha_{bx}=\min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]$

$\alpha_{bx} > 0.0$ 0.61 > 0.00 verified

$F_{b,Rd2x}$ = 214.85 [kN] Bearing resistance of a single bolt $F_{b,Rd2x}=k_{1x}*\alpha_{bx}*f_u*d*\sum t_i/\gamma_{M2}$

Direction y

k_{1y} = 2.50 Coefficient for calculation of $F_{b,Rd}$ $k_{1y}=\min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]$

$k_{1y} > 0.0$ 2.50 > 0.00 verified

α_{by} = 0.69 Coefficient for calculation of $F_{b,Rd}$ $\alpha_{by}=\min[e_2/(3*d_0), f_{ub}/f_u, 1]$

$\alpha_{by} > 0.0$ 0.69 > 0.00 verified

$F_{b,Rd2y}$ = 244.25 [kN] Bearing resistance of a single bolt $F_{b,Rd2y}=k_{1y}*\alpha_{by}*f_u*d*\sum t_i/\gamma_{M2}$

ULTIMATE LIMIT STATE

Bolt shear

F_{Ed} = -51.00 [kN] Shear force in a bolt $F_{Ed}=N_{Ed,pfle}/n_b$

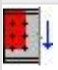
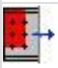
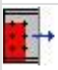
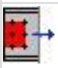
β_{Lf} = 1.00 Reduction factor for long connections $\beta_{Lf} = \max(0.75, \min(1; 1-(L-15*d)/(200*d)))$

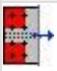
F_{Rd} = 173.72 [kN] Effective design capacity of a bolt $F_{Rd} = \min(F_{v,Rd}; F_{b,Rd1}; F_{b,Rd2})$

$|F_{Ed}| \le \beta_{Lf}*F_{Rd}$ |-51.00| < 173.72 verified (0.29)

VERIFICATION OF THE SECTION DUE TO BLOCK TEARING - [3.10]

BEAM

Nr	Model	A_{nv} [mm2]	A_{nt} [mm2]	V_0 [kN]	$V_{eff,Rd}$ [kN]	$\lvert V_0\rvert/V_{eff,Rd}$	Status
1		1757	1322	47.00 (*1)	506.25 (*)	0.09	verified
2		1322	6768	1.51 (*2)	2537.88 (**)	0.00	verified
3		1322	6768	1.51 (*2)	2537.88 (**)	0.00	verified
4		2643	980	1.51 (*2)	756.83 (**)	0.00	verified

Nr	Model	A_{nv} [mm2]	A_{nt} [mm2]	V_0 [kN]	$V_{eff,Rd}$ [kN]	$\lvert V_0\rvert/V_{eff,Rd}$	Status
5		2849	611	309.50 (*3)	1324.78 (***)	0.23	verified

(*1) $V_0 = V_{zEd1}$

(*2) $V_0 = N_{wEd}$

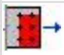
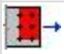
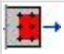
(*3) $V_0 = N_{fuEd}$

(*) $V_{effRd} = 0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

(**) $V_{effRd} = f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

(***) $V_{effRd} = 2*[f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}]$

Splice plate

Nr	Model	A_{nv} [mm2]	A_{nt} [mm2]	V_0 [kN]	$V_{eff,Rd}$ [kN]	$\lvert V_0\rvert/V_{eff,Rd}$	Status
1		2470	2420	47.00 (*1)	980.57 (*)	0.05	verified
2		2420	2470	1.51 (*2)	1464.24 (**)	0.00	verified
3		2420	2470	1.51 (*2)	1464.24 (**)	0.00	verified
4		4840	1980	1.51 (*2)	1768.16 (**)	0.00	verified

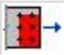
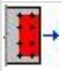
(*1) $V_0 = 0.5*V_{zEd1}$

(*2) $V_0 = 0.5*N_{wEd}$

(*) $V_{effRd} = 0.5*f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

(**) $V_{effRd} = f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

Upper external plate

Nr	Model	A_{nv} [mm2]	A_{nt} [mm2]	V_0 [kN]	$V_{eff,Rd}$ [kN]	$\lvert V_0\rvert/V_{eff,Rd}$	Status
1		2138	3225	295.20 (*1)	1702.30 (**)	0.17	verified
2		4275	2610	295.20 (*1)	1899.32 (**)	0.16	verified

(*1) $V_0 = N_{fueEd}$

(**) $V_{effRd} = f_u*A_{nt}/\gamma_{M2} + (1/\sqrt{3})*f_y*A_{nv}/\gamma_{M0}$

Verification of sections weakened by openings - [5.4]

Beam

A_t =	6181	[mm^2]	Area of tension zone of the gross section
$A_{t,net}$ =	5123	[mm^2]	Net area of the section in tension

$0.9*(A_{t,net}/A_t) \geq (f_y*\gamma_{M2})/(f_u*\gamma_{M0})$ 0.75 < 0.80

W =	1445274	[mm^3]	Elastic section modulus
W_{net} =	1445274	[mm^3]	Elastic section modulus

$W =$	1445274	[mm^3]	Elastic section modulus	
$M_{c,Rdnet} =$	397.45	[kN*m]	Design resistance of the section for bending	$M_{c,Rdnet} = W_{net}*f_{yp}/\gamma_{M0}$
$\lvert M_0\rvert \le M_{c,Rdnet}$		|100.00| < 397.45	verified	(0.25)
$A_v =$	3048	[mm^2]	Effective section area for shear	$A_v = h_p*t_p$
$A_{v,net} =$	2533	[mm^2]	Net area of a section effective for shear	$A_{vnet}=A_v-n_v*d_0*t_p$
$V_{pl,Rd} =$	483.97	[kN]	Design plastic resistance for shear	$V_{pl,Rd}=(A_v*f_{yp})/(\sqrt{3}*\gamma_{M0})$
$\lvert V_0\rvert \le V_{pl,Rd}$		|47.00| < 483.97	verified	(0.10)

SPLICE PLATE

$A_t =$	2009	[mm^2]	Area of tension zone of the gross section	
$A_{t,net} =$	1489	[mm^2]	Net area of the section in tension	
$0.9*(A_{t,net}/A_t) \ge (f_y*\gamma_{M2})/(f_u*\gamma_{M0})$		0.67 < 0.91		
$W =$	133333	[mm^3]	Elastic section modulus	
$W_{net} =$	120992	[mm^3]	Elastic section modulus	
$M_{c,Rdnet} =$	42.95	[kN*m]	Design resistance of the section for bending	$M_{c,Rdnet} = W_{net}*f_{yp}/\gamma_{M0}$
$\lvert M_0\rvert \le M_{c,Rdnet}$		|10.88| < 42.95	verified	(0.25)
$A_v =$	4000	[mm^2]	Effective section area for shear	$A_v = h_p*t_p$
$A_{v,net} =$	2960	[mm^2]	Net area of a section effective for shear	$A_{vnet}=A_v-n_v*d_0*t_p$
$V_{pl,Rd} =$	819.84	[kN]	Design plastic resistance for shear	$V_{pl,Rd}=(A_v*f_{yp})/(\sqrt{3}*\gamma_{M0})$
$\lvert V_0\rvert \le V_{pl,Rd}$		|47.00| < 819.84	verified	(0.06)

UPPER EXTERNAL PLATE

$A =$	4620	[mm^2]	Area of tension zone of the gross section	$A=h_{pi}*t_{pi}$
$A_{net} =$	3840	[mm^2]	Net cross-sectional area	$A_{net}=A-n_v*d_0*t_{pi}$
$N_{pl,Rd} =$	1640.10	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd}=A*f_y/\gamma_{M0}$
$N_{u,Rd} =$	1354.75	[kN]	Design ultimate resistance to normal force of the net section	$N_{u,Rd}=0.9*A_{net}*f_u/\gamma_{M2}$
$F_{Ed} =$	295.20	[kN]		$F_{Ed} = N_{Ed,pfue}$
$\lvert F_{Ed}\rvert \le N_{u,Rd}$		|295.20| < 1354.75	verified	(0.22)
$\lvert F_{Ed}\rvert \le N_{pl,Rd}$		|295.20| < 1640.10	verified	(0.18)

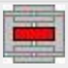
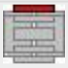
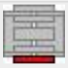
LOWER EXTERNAL PLATE

$A =$	4620	[mm^2]	Area of tension zone of the gross section	$A=h_{pi}*t_{pi}$
$A_{net} =$	3840	[mm^2]	Net cross-sectional area	$A_{net}=A-n_v*d_0*t_{pi}$
$N_{pl,Rd} =$	1640.10	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd}=A*f_y/\gamma_{M0}$
$N_{u,Rd} =$	1354.75	[kN]	Design ultimate resistance to normal force of the net section	$N_{u,Rd}=0.9*A_{net}*f_u/\gamma_{M2}$
$F_{Ed} =$	-291.71	[kN]		$F_{Ed} = N_{Ed,pfli}$
$\lvert F_{Ed}\rvert \le N_{u,Rd}$		|-291.71| < 1354.75	verified	(0.22)
$\lvert F_{Ed}\rvert \le N_{pl,Rd}$		|-291.71| < 1640.10	verified	(0.18)

$\lvert F_{Ed}\rvert \le N_{u,Rd}$	|-291.71| < 1354.75	verified	(0.22)

LEFT SIDE

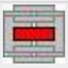
Axial force

Plate	A_i [mm2]	EQUIVALENT FORCES N_i [kN]	EQUIVALENT FORCES $N_i(M_{y,Ed})$ [kN]	Resultant force $N_{Ed,i}$ [kN]
	A_{pw}= 4000	0.00	-	**$N_{Ed,pw}$= 0.00**
	A_{pfue}= 4620	0.00	0.00	**$N_{Ed,pfue}$= 0.00**
	A_{pfle}= 4620	0.00	0.00	**$N_{Ed,pfle}$= 0.00**

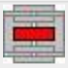
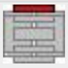
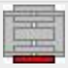
$N_i=(N_{Ed}*A_i)/(A_{wp}+A_{pfue}+A_{pfle})$

$N_{Ed,i} = N_i+N_i(M_{y,Ed})$

Shear force Z

Plate	A_i [mm2]	$V_{zEd,i}$ [kN]
	$A_{z,pw}$= 4000	$V_{z,Ed,pw}$= 0.00

Bending moment Y

Plate	$I_{y,i}$ [mm4]	EQUIVALENT FORCES $M_{y,i}$ [kN*m]	Resultant force $M_{y,Ed,i}$ [kN*m]
	$I_{y,pw}$= 13333333	0.00	**$M_{y,Ed,pw}$= 0.00**
	$I_{y,pfue}$= 120512019	0.00	-
	$I_{y,pfle}$= 120512019	0.00	-

$M_{y,i}=(M_{y,Ed}*I_{y,i})/(I_{pw}+I_{pfue}+I_{pfle})$

BOLTS CONNECTING A SPLICE PLATE WITH THE BEAM WEB

BOLT CAPACITIES

$F_{v,Rd}$ =	173.72	[kN]	Shear bolt resistance in the unthreaded portion of a bolt	$F_{v,Rd}= 0.6*f_{ub}*A_v*m/\gamma_{M2}$

Bolt bearing on the beam

Direction x

k_{1x} =	2.50	Coefficient for calculation of $F_{b,Rd}$	$k_{1x} = \min[2.8*(e_1/d_0)-1.7,\ 1.4*(p_1/d_0)-1.7,\ 2.5]$
$k_{1x} > 0.0$	2.50 > 0.00	verified	
α_{bx} =	0.80	Coefficient for calculation of $F_{b,Rd}$	$\alpha_{bx}=\min[e_2/(3*d_0),\ p_2/(3*d_0)-0.25,\ f_{ub}/f_u,\ 1]$
$\alpha_{bx} > 0.0$	0.80 > 0.00	verified	

$F_{b,Rd1x}$ =	163.73	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1x}=k_{1x}*\alpha_{bx}*f_u*d*\sum t_i/\gamma_{M2}$

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_2/d_0)-1.7, 1.4*(p_2/d_0)-1.7, 2.5]
$k_{1z} > 0.0$			2.50 > 0.00 verified	
α_{bz} =	1.00		Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
$\alpha_{bz} > 0.0$			1.00 > 0.00 verified	
$F_{b,Rd1z}$ =	204.34	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1z}$=k_{1z}*α_{bz}*f_u*d*∑t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2.34		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
$k_{1x} > 0.0$			2.34 > 0.00 verified	
α_{bx} =	0.64		Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_2/(3*d_0), p_2/(3*d_0)-0.25, f_{ub}/f_u, 1]
$\alpha_{bx} > 0.0$			0.64 > 0.00 verified	
$F_{b,Rd2x}$ =	282.05	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction z

k_{1z} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1z}=min[2.8*(e_2/d_0)-1.7, 1.4*(p_2/d_0)-1.7, 2.5]
$k_{1z} > 0.0$			2.50 > 0.00 verified	
α_{bz} =	0.48		Coefficient for calculation of $F_{b,Rd}$	α_{bz}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
$\alpha_{bz} > 0.0$			0.48 > 0.00 verified	
$F_{b,Rd2z}$ =	226.15	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2z}$=k_{1z}*α_{bz}*f_u*d*∑t_i/γ_{M2}

ULTIMATE LIMIT STATE

Bolt shear

$F_{x,Ed}$ =	0.00	[kN]	Design total force in a bolt on the direction x	
$F_{z,Ed}$ =	0.00	[kN]	Design total force in a bolt on the direction z	
F_{Ed} =	0.00	[kN]	Resultant shear force in a bolt	$F_{Ed} = \sqrt{(F_{x,Ed}^2 + F_{z,Ed}^2)}$
$F_{Rd,x}$ =	163.73	[kN]	Effective design capacity of a bolt on the direction x	F_{Rdx}=min($F_{bRd1,x}$, $F_{bRd2,x}$)
$F_{Rd,z}$ =	204.34	[kN]	Effective design capacity of a bolt on the direction z	F_{Rdz}=min($F_{bRd1,z}$, $F_{bRd2,z}$)
$\lvert F_{x,Ed}\rvert \le F_{Rd,x}$			|0.00| < 163.73 verified	(0.00)
$\lvert F_{z,Ed}\rvert \le F_{Rd,z}$			|0.00| < 204.34 verified	(0.00)
$F_{Ed} \le F_{v,Rd}$			0.00 < 173.72 verified	(0.00)

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM TOP FLANGE

BOLT CAPACITIES

$F_{v,Rd}$ =	173.72	[kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam flange

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]

$k_{1x} > 0.0$		2.50 > 0.00	verified	
α_{bx} =	0.78	Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
$\alpha_{bx} > 0.0$		0.78 > 0.00	verified	
$F_{b,Rd1x}$ =	246.54 [kN]	Bearing resistance of a single bolt		$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} =	2.50	Coefficient for calculation of $F_{b,Rd}$		k_{1y} = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
$k_{1y} > 0.0$		2.50 > 0.00	verified	
α_{by} =	0.68	Coefficient for calculation of $F_{b,Rd}$		α_{by}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
$\alpha_{by} > 0.0$		0.68 > 0.00	verified	
$F_{b,Rd1y}$ =	214.55 [kN]	Bearing resistance of a single bolt		$F_{b,Rd1y}$=k_{1y}*α_{by}*f_u*d*∑t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2.50	Coefficient for calculation of $F_{b,Rd}$		k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
$k_{1x} > 0.0$		2.50 > 0.00	verified	
α_{bx} =	0.61	Coefficient for calculation of $F_{b,Rd}$		α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
$\alpha_{bx} > 0.0$		0.61 > 0.00	verified	
$F_{b,Rd2x}$ =	214.85 [kN]	Bearing resistance of a single bolt		$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} =	2.50	Coefficient for calculation of $F_{b,Rd}$		k_{1y}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
$k_{1y} > 0.0$		2.50 > 0.00	verified	
α_{by} =	0.69	Coefficient for calculation of $F_{b,Rd}$		α_{by}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
$\alpha_{by} > 0.0$		0.69 > 0.00	verified	
$F_{b,Rd2y}$ =	244.25 [kN]	Bearing resistance of a single bolt		$F_{b,Rd2y}$=k_{1y}*α_{by}*f_u*d*∑t_i/γ_{M2}

ULTIMATE LIMIT STATE

Bolt shear

F_{Ed} =	0.00 [kN]	Shear force in a bolt		F_{Ed}=$N_{Ed,pfue}$/n_b
β_{Lf} =	1.00	Reduction factor for long connections		β_{Lf} = max(0.75, min(1; 1-(L-15*d)/(200*d)))
F_{Rd} =	173.72 [kN]	Effective design capacity of a bolt		F_{Rd} = min($F_{v,Rd}$; $F_{b,Rd1}$; $F_{b,Rd2}$)
$\lvert F_{Ed}\rvert \le \beta_{Lf}$*$F_{Rd}$		|0.00| < 173.72	verified	(0.00)

BOLTS CONNECTING A FLANGE PLATE WITH THE BEAM BOTTOM FLANGE

BOLT CAPACITIES

$F_{v,Rd}$ =	173.72 [kN]	Shear resistance of the shank of a single bolt	$F_{v,Rd}$= 0.6*f_{ub}*A_v*m/γ_{M2}

Bolt bearing on the beam flange

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00 verified	
α_{bx} =	0.78		Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0			0.78 > 0.00 verified	
$F_{b,Rd1x}$ =	246.54	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1y} = min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1y} > 0.0			2.50 > 0.00 verified	
α_{by} =	0.68		Coefficient for calculation of $F_{b,Rd}$	α_{by}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{by} > 0.0			0.68 > 0.00 verified	
$F_{b,Rd1y}$ =	214.55	[kN]	Bearing resistance of a single bolt	$F_{b,Rd1y}$=k_{1y}*α_{by}*f_u*d*∑t_i/γ_{M2}

Bolt bearing on the plate

Direction x

k_{1x} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1x}=min[2.8*(e_2/d_0)-1.7, 2.5]
k_{1x} > 0.0			2.50 > 0.00 verified	
α_{bx} =	0.61		Coefficient for calculation of $F_{b,Rd}$	α_{bx}=min[e_1/(3*d_0), p_1/(3*d_0)-0.25, f_{ub}/f_u, 1]
α_{bx} > 0.0			0.61 > 0.00 verified	
$F_{b,Rd2x}$ =	214.85	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2x}$=k_{1x}*α_{bx}*f_u*d*∑t_i/γ_{M2}

Direction y

k_{1y} =	2.50		Coefficient for calculation of $F_{b,Rd}$	k_{1y}=min[2.8*(e_1/d_0)-1.7, 1.4*(p_1/d_0)-1.7, 2.5]
k_{1y} > 0.0			2.50 > 0.00 verified	
α_{by} =	0.69		Coefficient for calculation of $F_{b,Rd}$	α_{by}=min[e_2/(3*d_0), f_{ub}/f_u, 1]
α_{by} > 0.0			0.69 > 0.00 verified	
$F_{b,Rd2y}$ =	244.25	[kN]	Bearing resistance of a single bolt	$F_{b,Rd2y}$=k_{1y}*α_{by}*f_u*d*∑t_i/γ_{M2}

ULTIMATE LIMIT STATE

Bolt shear

F_{Ed} =	0.00	[kN]	Shear force in a bolt	F_{Ed}=$N_{Ed,pfle}$/n_b
β_{Lf} =	1.00		Reduction factor for long connections	β_{Lf} = max(0.75, min(1; 1-(L-15*d)/(200*d)))
F_{Rd} =	173.72	[kN]	Effective design capacity of a bolt	F_{Rd} = min($F_{v,Rd}$; $F_{b,Rd1}$; $F_{b,Rd2}$)
|F_{Ed}| ≤ β_{Lf}*F_{Rd}			|0.00| < 173.72 verified	(0.00)

VERIFICATION OF SECTIONS WEAKENED BY OPENINGS - [5.4]

BEAM

UPPER EXTERNAL PLATE

A =	4620	[mm^2]	Area of tension zone of the gross section	A=h_{pi}*t_{pi}
A_{net} =	3840	[mm^2]	Net cross-sectional area	A_{net}=A-n_v*d_0*t_{pi}

UPPER EXTERNAL PLATE

A =	4620	[mm^2]	Area of tension zone of the gross section	$A=h_{pi}*t_{pi}$
$N_{pl,Rd}$ =	1640.10	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd}=A*f_y/\gamma_{M0}$
$N_{u,Rd}$ =	1354.75	[kN]	Design ultimate resistance to normal force of the net section	$N_{u,Rd}=0.9*A_{net}*f_u/\gamma_{M2}$
F_{Ed} =	0.00	[kN]		$F_{Ed} = N_{Ed,pfue}$

$\lvert F_{Ed}\rvert \le N_{u,Rd}$	|0.00| < 1354.75	verified	(0.00)
$\lvert F_{Ed}\rvert \le N_{pl,Rd}$	|0.00| < 1640.10	verified	(0.00)

LOWER EXTERNAL PLATE

A =	4620	[mm^2]	Area of tension zone of the gross section	$A=h_{pi}*t_{pi}$
A_{net} =	3840	[mm^2]	Net cross-sectional area	$A_{net}=A-n_v*d_0*t_{pi}$
$N_{pl,Rd}$ =	1640.10	[kN]	Design plastic resistance of the gross section	$N_{pl,Rd}=A*f_y/\gamma_{M0}$
$N_{u,Rd}$ =	1354.75	[kN]	Design ultimate resistance to normal force of the net section	$N_{u,Rd}=0.9*A_{net}*f_u/\gamma_{M2}$
F_{Ed} =	0.00	[kN]		$F_{Ed} = N_{Ed,pfli}$

$\lvert F_{Ed}\rvert \le N_{u,Rd}$	|0.00| < 1354.75	verified	(0.00)
$\lvert F_{Ed}\rvert \le N_{pl,Rd}$	|0.00| < 1640.10	verified	(0.00)

Connection conforms to the code Ratio 0.30