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(Legge n° 1158 del 17 dicembre 1971, modificata dal D.Lgs. n°114 del 24 aprile 2003)

PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

EUROLINK S.C.p.A.

IMPREGILO S.p.A. (MANDATARIA)
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INDICE

INDICE ... 3
1 Introduction ... 7
1.1 Outline ... 7
2 Design References ... 8
2.1 Design Specifications ... 8
2.2 Design Codes ... 8
2.3 Drawings ... 9
2.3.1 Complementary Reports ... 10
3 Materials ... 10
3.1 Structural Steel ... 10
3.2 High Strength Bolts ... 11
3.3 Welding Consumables ... 12
4 Design Principles ... 12
5 Tower Legs ... 13
5.1 Longitudinal Elements ... 13
5.1.1 Longitudinal Plates and Stiffeners ... 13
5.1.1.1 Section Properties ... 14
5.1.1.2 Global Design Loads ... 16
5.1.1.3 Global Stability of the Tower Legs ... 19
5.1.1.4 Capacity of the Cross Section ... 23
5.1.1.5 Calculation of Stresses and Utilization Ratios ... 35
5.1.1.6 Overall Verification for Entire Tower for Single Load Combination ... 42
5.1.1.7 Verification for All Load Combinations ... 47
5.1.1.8 Verification for the Envelope of All Time-History Seismic Results ... 58
5.1.1.9 Shear Stresses ... 65
5.1.2 Joints and Splices ... 69
5.2 Transverse Elements ... 83
5.2.1 Transverse Stiffeners and Type 1 Diaphragms ... 83
5.2.2 Cross Beam to Tower Leg Connection – Type 2 and 3 Diaphragms ... 112
5.2.3 Deck Buffer Connections ... 167
5.3 Tower Leg Segment Detailed Finite Element Analysis ... 198

5.3.1 Introduction ... 198
5.3.2 Finite Element Model Description ... 198
5.3.3 Details of the Finite Element Model ... 201
5.3.4 Application of Loads ... 202
5.3.5 Material Properties ... 202
5.3.6 Run cases ... 205
5.3.6.1 Imperfection case ... 205
5.3.6.2 Multi-Linear Cases ... 207
5.3.6.3 Bilinear Cases (Effect of residual stresses) ... 218
5.3.6.4 Effect of Model Restraint to the Plate A Longitudinal Stiffener ... 220
5.3.7 Conclusions ... 221
5.3.8 Assessment for the Envelope of Seismic Time-History Analysis Force Effects ... 221
5.4 Fatigue ... 237
5.5 Accidental Load Scenarios ... 240
5.5.1 Design Scenarios – Fire ... 240
5.5.2 Evaluation Scenarios ... 245
5.5.2.1 Impact ... 245
5.5.2.2 Explosion ... 258
6 Transverse Stiffener and Diaphragm Plate Buckling Analysis / Longitudinal to Transverse Stiffener Connections ... 261
6.1 Introduction ... 261
6.2 Task 1: Removal of Tab Plates ... 261
6.2.1 Background ... 261
6.2.2 Model Description ... 264
6.2.3 Loadings ... 266
6.2.4 Analysis Results Summary ... 266
6.2.5 Verification of Resistance ... 268
6.3 Task 2: Stability of Transverse Stiffener Flanges ... 268
6.3.1 Background ... 268
6.3.2 Methodology for Sizing Transverse Stiffener Flanges ... 269
6.3.3 Model and Loading Description ... 269
6.3.4 Analysis Result Summary ... 270
6.3.5 Verification of Resistance ... 272

6.4 Task 3: Stability of Triangular Diaphragm Plates 273
6.4.1 Background 273
6.4.2 Model Description........ 273
6.4.3 Loadings........ 275
6.4.4 Analysis Result Summary 276
6.4.5 Verification of Resistance........ 281

1 Introduction

This report presents design calculations for the tower legs. The design is based on that shown in the Tender Design. In this project phase it was found advantageous to introduce the following changes to the tender design:

- The tower height was increased from 382.6 m to 399 m to compensate for the increase in deck weight;
- Flat plate longitudinal stiffeners replace T-shaped longitudinal stiffeners in the tower legs and cross beams;
- The tower leg transverse stiffener arrangement was revised to simplify fabrication and assembly of the tower leg segments;
- The tower leg transverse diaphragm arrangement was revised to eliminate unnecessary material; and
- Specifications of tuned mass dampers were modified based on the results of wind tunnel testing.

Calculations are typically based on the global IBDAS model version 3.3f.

1.1 Outline

This report is organized into the following sections:

- Section 1 includes this introduction and outline;
- Section 2 provides a list of reference materials, including design specifications, design codes, reference drawings and complementary reports;
- Section 3 provides details of the materials used in the tower design;
- Section 4 provides a reference to the reports describing the design principles used in the tower design;
- Section 5 provides design calculations for the various tower leg components, including a section describing the detailed finite element analysis of a full tower leg segment. To allow

for easy cross referencing between tower design reports, the calculations are presented in the same order as the components are described in CG.10.00-P-RX-D-P-SV-T4-00-00-00-00-01 “Specialist Technical Design Report, Towers;”

- Section 6 provides a description of the detailed finite element analysis that was completed to support the removal of the tab plates that connected the longitudinal stiffeners to the transverse stiffeners in the general concept submission, to size the transverse stiffener flanges and to determine the adequate thickness of the triangular diaphragm plates between tower leg plates B, C, E, F and H; and

2 Design References

2.1 Design Specifications

CG.10.00-P-RG-D-P-GE-00-00-00-00-00-02-A - “Design Basis, Structural, Annex,” COWI 2010

GCG.F.05.03 “Design Development – Requirements and Guidelines,” Stretto di Messina, 2004 October 22.

GCG.G.03.02 “Structural Steel Works and Protective Coatings,” Stretto di Messina, 2004 July 30.

2.2 Design Codes

“Norme tecniche per le costruzioni,” 2008 (NTC08).

EN 1991 Eurocode 1: Actions on Structures – Part 2: Traffic loads on bridges

EN 1993 Eurocode 3: Design of Steel Structures – Part 1-1: General rules and rules for buildings

EN 1993 Eurocode 3: Design of Steel Structures – Part 1-5: Plated structural elements

EN 1993 Eurocode 3: Design of Steel Structures – Part 1-8: Design of joints

EN 1993 Eurocode 3: Design of Steel Structures – Part 1-9: Fatigue

EN 1993 Eurocode 3: Design of Steel Structures – Part 2: Steel Bridges

Rete Ferroviaria Italia - Istruzione No. 44F “Verifiche a fatica dei ponti ferroviari”

2.3 Drawings

The reference tower design drawings for this report are listed in Table 2-1.

Drawing Title	Drawing Number
Tower Sicilia - General Arrangement	CG.10.00-P-AX-D-P-SV-T4-TS-00-00-00-01_0
Typical - Leg - Cross Section & Vertical Joints	CG.10.00-P-WX-D-P-SV-T4-TO-00-00-00-01_0
Tower Sicilia - Leg - Sections & Plate Thicknesses	CG.10.00-P-WX-D-P-SV-T4-TS-00-00-00-01_0
Typical - Leg - Cross Diaphragms	CG.10.00-P-AX-D-P-SV-T4-TO-00-D0-00-01_0
Typical - Leg - Cross Diaphragms, Details	CG.10.00-P-BX-D-P-SV-T4-TO-00-D0-00-01_0
Typical - Leg - Horizontal Joints	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-01_0
Typical - Leg - Horizontal Joints, Details	CG.10.00-P-BX-D-P-SV-T4-TO-00-00-00-01_0
Typical - Cross Beam no. 1	CG.10.00-P-AX-D-P-SV-T4-TO-T0-00-00-01_0
Typical - Cross Beam no. 2	CG.10.00-P-AX-D-P-SV-T4-TO-T0-00-00-02_0
Typical - Cross Beam no. 3	CG.10.00-P-AX-D-P-SV-T4-TO-T0-00-00-03_0
Typical - Cross Beams - Details	CG.10.00-P-BX-D-P-SV-T4-TO-T0-00-00-01_0
Typical - Cross Beam Connection to Tower leg	CG.10.00-P-WX-D-P-SV-T4-TO-T0-00-00-02_0
Typical - Connection from Girder	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-03_0
Typical - Base Section 1	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-04_0
Typical - Base Section 2	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-05_0
Typical - Base Section, Details	CG.10.00-P-BX-D-P-SV-T4-TO-00-00-00-02_0
Typical - Top Section	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-06_0
Typical - Tuned Mass Dampers - Support Structure	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-07_0
Typical - Tuned Mass Dampers	CG.10.00-P-AX-D-P-SV-T4-TO-00-00-00-08_0
Typical - Leg - Steelwork Modifications 1	CG.10.00-P-BX-D-P-SV-T4-TO-00-00-00-04_0
Typical - Leg - Steelwork Modifications 2	CG.10.00-P-BX-D-P-SV-T4-TO-00-00-00-05_0
Typical - Cross Beams - Steelwork Modifications	CG.10.00-P-BX-D-P-SV-T4-TO-T0-00-00-06_0
Tower Calabria - General Arrangement	CG.10.00-P-AX-D-P-SV-T4-TC-00-00-00-01_0
Tower Calabria - Leg - Sections & Plate Thicknesses	CG.10.00-P-WX-D-P-SV-T4-TC-00-00-00-01_0

Table 2-1: Reference tower drawings.

2.3.1 Complementary Reports

The tower design reports listed in Table 2-2 provide supplementary information about the tower design principles and verifications.

Report Title	Report Number
Specialist Technical Design Report, Towers	CG.10.00-P-RX-D-P-SV-T4-00-00-00-00-01
General Design Principles	CG.10.00-P-RG-D-P-SV-T4-00-00-00-00-01
Design Report - Cross Beams	CG.10.00-P-CL-D-P-SV-T4-00-00-00-00-02
Design Report - Tower Base	CG.10.00-P-CL-D-P-SV-T4-00-00-00-00-03

Table 2-2: Reference tower design reports.

3 Materials

The mechanical properties of the tower construction materials are described in this section.

3.1 Structural Steel

Tower structural components are generally fabricated from Grade S460 ML structural steel with the exception of: the hot-rolled circular hollow sections comprising the cross beam internal bracing members, the tower base plate and the base anchorage stiffening plates, which are fabricated from Grade S355 ML structural steel. All structural steels shall be produced in accordance with EN 10025-4. The steels are assumed to have the mechanical properties listed in Table 3-1, in accordance with NTC08 Section 11.3.4.1. As an exception to the standard requirements of NTC08 and EN 10025-4 the mechanical properties of the steel shall not vary with material thickness for thicknesses up to 110 mm for S460ML steel and up to 150 mm for S355ML steel. The feasibility of the production of steel with the required properties has been confirmed.

Grade	Yield Strength, f_{yk} (MPa)	Tensile Strength, f_{tk} (MPa)
S 355 ML	355	470
S 460 ML	460	540

Table 3-1: Structural steel mechanical properties for thicknesses up to 110 mm for S460ML steel and up to 150 mm for S355ML steel.

All structural steel is also assumed to have the following properties, in accordance with NTC08 Section 11.3.4.1:

- Elastic modulus: $E = 210{,}000$ MPa
- Poisson's ratio: $\nu = 0.3$
- Shear modulus: $G = 80{,}770$ MPa
- Coefficient of thermal expansion: $\alpha = 12 \times 10^{-6} / °C$
- Density: $\rho = 7{,}850$ kg/m^3

The material partial factors (safety coefficients) used to verify structural steel elements are in accordance with NTC08 Sections 4.2.4.1.1, 4.2.4.1.4 and are listed in Table 3-2.

Verification	Partial Factor
Resistance of Class 1, 2, 3 and 4 sections	$\gamma_{M0} = 1.05$
Resistance to instability of members in road and rail bridges	$\gamma_{M1} = 1.10$
Resistance to fracture of sections under tension (weakened by holes)	$\gamma_{M2} = 1.25$
Fatigue resistance (useful fatigue life criterion with significant failure consequences)	$\gamma_{mf} = 1.35$

Table 3-2: Material partial factors for structural steel.

3.2 High Strength Bolts

High strength structural bolts of Grade 8.8 or Grade 10.9, produced in accordance with EN ISO 898, are used for all bolted connections and splices. Grade 8.8 bolts are used for connections of all non-structural components to the towers and Grade 10.9 bolts are used for the tower leg construction joint splices (except for the skin plates splices, which are welded). High strength bolts are assumed to have the mechanical properties listed in Table 3-3, in accordance with NTC08 Section 11.3.4.6.1 (except for the Grade 8.8 yield strength, which is incorrectly stated in NTC08).

Grade	Yield Strength, f_{yb} (MPa)	Tensile Strength, f_{tb} (MPa)
8.8	640	800
10.9	900	1000

Table 3-3: Structural bolt mechanical properties.

The material partial factors (safety coefficients) used to verify bolted connections and splices are in accordance with NTC08 Section 4.2.8.1.1 and are listed in Table 3-4.

Verification	Partial Factor
Resistance to bolt shear	$\gamma_{M2} = 1.25$
Resistance to bolt tension	
Resistance to bearing on plates	
ULS slip resistance	$\gamma_{M3} = 1.25$
SLS slip resistance	$\gamma_{M3} = 1.15$
Bolt preload force	$\gamma_{M7} = 1.10$

Table 3-4: Material partial factors for bolted connections and splices.

3.3 Welding Consumables

Welding consumables shall comply with the requirements of EN 1993-1-8 Section 4.2.

Welding procedures shall be selected so as to not reduce the properties of the thermo-mechanically processed plates.

The material partial factor, $\gamma_{M2} = 1.25$, used to verify welded connections and splices is in accordance with NTC08 Section 4.2.8.1.1.

4 Design Principles

The design principles are primarily described in CG.10.00-P-RG-D-P-SV-T4-00-00-00-00-01 “General Design Principles.”

Summaries and discussions of verification results are provided in CG.10.00-P-RX-D-P-SV-T4-00-00-00-00-01 “Specialist Technical Design Report, Towers.”

5 Tower Legs

5.1 Longitudinal Elements

5.1.1 Longitudinal Plates and Stiffeners

The design of the longitudinal steel in the tower legs is based on EN 1993-1-5 and EN 1993-1-1 as described in CG.10.00-P-RX-D-P-SV-T4-00-00-00-00-01 “Specialist Technical Design Report.” The general design procedure for the longitudinal steel in the tower legs is described in CG.10.00-P-RG-D-P-SV-T4-00-00-00-00-01 “General Design Principles.” The following provides a further description of the design procedure for the tower legs by means of sample calculations for a typical tower segment. A summary of maximum utilization ratios are also presented for all segments and load cases.

The following sample calculations apply to Sicilia tower leg segment 5. The following figures illustrate the location of the segment and the relevant plate dimensions.

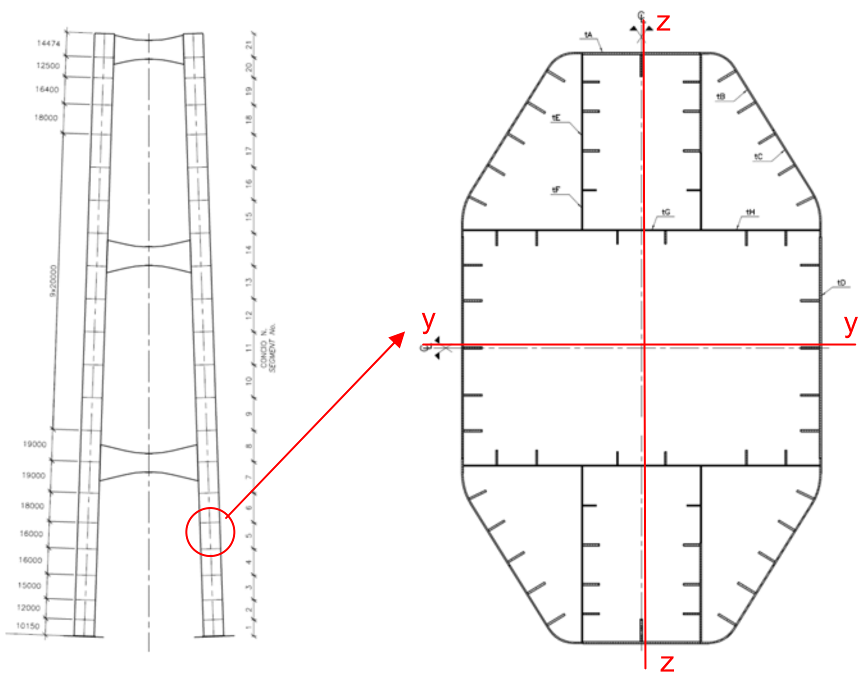


Plate and Stiffener Dimensions for Sicilia Tower Segment Five

Plate	tA	tB	tC	tD	tE	tF	tG	tH
Plate Thickness	110	90	45	40	60	35	35	40
Stiffener Dimension	750 x 75	675 x 68	625 x 63	600 x 60	675 x 68	500 x 50	450 x 45	450 x 45

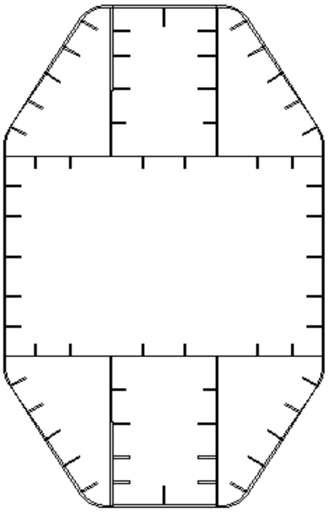
5.1.1.1 Section Properties

For the cross section and plate dimensions shown, the gross cross section properties are calculated and confirmed using AutoCAD. The calculated segment 5 section properties are as follows:

Cross Section	5
Analysis Section	10
Elevation	87

			Plates	Stiffeners	Total	
			Gross	Gross	Gross	Name
Gross Section Properties	A	(mm^2)	5.64E+06	2.21E+06	**7.841E+06**	*Ag, Ae*
	y-coord.	(mm)	0	0	**0**	*yg,ye*
	z-coord.	(mm)	10000	10000	**10000**	*zg, ze*
	I_y	(mm^4)	8.64E+14	3.18E+14	1.182E+15	
	I_z	(mm^4)	7.41E+13	3.03E+13	1.044E+14	
	I_Y	(mm^4)			**3.983E+14**	
	I_Z	(mm^4)			**1.044E+14**	
	I_{yz}	(mm^4)	**1.78E+14**	**7.17E+13**		
	I_{ZY}	(mm^4)			0.000E+00	
	Principal Axis					
	Angle	(rad)			**0.000E+00**	
	I_{1_Y}	(mm^4)			**3.983E+14**	*IgY, IeY*
	I_{2_Z}	(mm^4)			**1.044E+14**	*IgZ, IeZ*

The calculation of section properties shown above is performed in the design spreadsheet and uses simplifying assumptions at the curved corners. Therefore, for segment5, AutoCAD was used to verify the calculations and quantify any differences related to the section properties. The following figure shows the AutoCAD drawing of the cross section and corresponding section properties.



Segment 5

```
Area:                        7831626.9609
Perimeter:                   285184.6414
Bounding box:           X: -6000.0000  --  6000.0000
                        Y: -10000.0000  --  10000.0000
Centroid:               X: 0.0000
                        Y: 0.0000
Moments of inertia:     X: 3.9848E+14
                        Y: 1.0623E+14
Product of inertia:    XY: 0.1250
Radii of gyration:      X: 7133.0760
                        Y: 3682.9199
Principal moments and X-Y directions about centroid:
                        I: 1.0623E+14 along [0.0000 1.0000]
                        J: 3.9848E+14 along [-1.0000 0.0000]
```

From the AutoCAD output, the section properties are:

A_{gross} = 7.83 m^2

$I_{y,gross}$ = 398.5 m^4

$I_{z,gross}$ = 106.2 m^4

Therefore, it is concluded that the approximate spreadsheet calculations are sufficiently accurate for use in the general design of the tower legs.

The following tables show the section properties for all tower segments for the Sicilia and Calabria towers.

Sicilia Tower Section Properties

Tower Segment	Area m^2	Iy m^4	Iz m^4
1	10.91	516	157
2	10.02	488	141
3	9.23	461	129
4	8.58	434	118
5	7.84	398	104
6	7.45	362	103
7	7.53	324	114
8	7.88	311	131
9	7.45	302	120
10	7.09	308	107
11	6.98	310	102
12	7.19	311	108
13	7.52	327	115
14	7.88	351	119
15	7.72	366	111
16	7.50	371	102
17	7.39	369	100
18	7.22	342	101
19	7.14	300	111
20	7.43	302	116
21	7.71	313	119

Calabria Tower Section Properties

Tower Segment	Area m^2	Iy m^4	Iz m^4
1	9.82	466.1	147.7
2	9.05	439.5	134.0
3	8.34	406.6	122.2
4	7.83	378.7	114.2
5	7.29	348.0	103.2
6	6.87	303.4	102.7
7	7.08	291.5	112.3
8	7.68	305.2	129.1
9	7.15	294.2	116.6
10	6.93	305.2	104.4
11	6.91	320.6	97.1
12	7.05	317.7	103.6
13	7.35	317.0	112.5
14	7.54	328.6	115.6
15	7.33	342.4	106.0
16	7.17	343.9	99.6
17	6.94	331.2	94.7
18	6.80	312.4	97.4
19	6.73	283.7	105.8
20	7.09	280.6	114.0
21	7.36	298.8	111.3

5.1.1.2 Global Design Loads

For the tower leg design the governing load combinations are ULS seismic and ULS wind consisting of the following load components:

ULS 3: μPP + (0.9/1.5)PN + (1.1)QA + (1.0)VV + (0/1)VT

ULS 7: μPP + (0.9/1.5)PN + (1.1)QA + (1.0)VS + (0/1)VT

From the global IBDAS model the sectional forces matrix corresponding to a given ULS combination are provided at the both the top and bottom of each tower segment. The sectional force effects matrix provides the maximum and minimum loads for each force effect and the concurrent values for the off-diagonal effects. To ensure that the most critical loading patterns were considered, additional linear combinations were formulated to find the critical case for the stresses at the extreme fibers of the cross section. The linear combination cases were specified to maximize the loading for the following equations.

$$L001:\left(\frac{N_s}{EA}\right)-\left[\frac{(M_y\cdot 10m)}{EI_y}\right]-\left[\frac{(M_z\cdot 2m)}{EI_z}\right]$$

$$L002:\left(\frac{N_s}{EA}\right)-\left[\frac{(M_y\cdot 4m)}{EI_y}\right]-\left[\frac{(M_z\cdot 6m)}{EI_z}\right]$$

The seismic loads are based on time-history analysis results (average of 8 inputs), which require post processing of the global IBDAS output. Therefore, for the ULS seismic load combination, the seismic time-history results are added manually to the other load components in the ULS load combination. Similarly, the forces due to uniform temperature loading are provided separately and added in manually to account for the behaviour of the bilinear buffers.

For the example tower segment 5, the critical sets of forces are for ULS combination 7 and are the maximum forces at the bottom of the segment (EL +71). The following tables show the section force effects matrix for tower segment 5 under the governing ULS seismic load combination including the time-history and temperature effects.

Sectional forces for IBDAS standard load combination (does not include time history or temp)

Case	Criteria	Ns[MN]	My[MNm]	Mz[MNm]	Vy[MN]	Vz[MN]	Mt[MNm]	z[m]
6903	min NS	-1869	-30	-1	1	2	18	71
6903	max NS	-1090	-2840	-8	1	-4	-18	71
6903	min MY	-1095	-3838	-8	1	-7	31	71
6903	max MY	-1843	1079	-9	0	5	-27	71
6903	min MZ	-1822	167	-55	-1	4	92	71
6903	max MZ	-1271	-2361	32	2	-5	-104	71
6903	min VY	-1743	-192	-31	-1	4	106	71
6903	max VY	-1350	-1999	8	3	-5	-119	71
6903	min L001	-1857	1057	2	1	5	-29	71
6903	max L001	-1095	-3838	-8	1	-7	31	71
6903	min L002	-1860	1032	3	1	5	-30	71
6903	max L002	-1095	-3838	-8	1	-7	31	71

Sectional forces from Time History Results - ULS 3 - Longitudinal Dominate

LIMIT STATE NUMBER (ULS=3, SILS=4), NOT COMBINATION NUMBER

Case	Criteria	NS	MY	MZ	VY	VZ	MT	z[m]
L-3-L	min NS	-301.0	2323.0	264.0	13.0	26.0	5.0	71
L-3-L	max NS	259.0	-1117.0	-29.0	6.0	1.0	-20.0	71
L-3-L	min MY	87.0	-3707.0	23.0	3.0	-49.0	-10.0	71
L-3-L	max MY	-72.0	3699.0	-21.0	1.0	66.0	-8.0	71
L-3-L	min MZ	13.0	1235.0	-486.0	-12.0	12.0	2.0	71
L-3-L	max MZ	-103.0	-519.0	477.0	5.0	-13.0	7.0	71
L-3-L	min VY	-6.0	417.0	-182.0	-31.0	3.0	8.0	71
L-3-L	max VY	-3.0	172.0	135.0	29.0	24.0	-16.0	71
L-3-L	min L001	-242.0	3281.0	172.0	9.0	35.0	7.0	71
L-3-L	max L001	140.0	-3612.0	35.0	5.0	-39.0	-7.0	71
L-3-L	min L002	-291.0	2782.0	289.0	13.0	32.0	2.0	71
L-3-L	max L002	175.0	-3169.0	-139.0	-5.0	-23.0	-22.0	71

Combined Sectional forces [IBDAS standard load comb + time history] * does not include unif. temp.

Case	Criteria	Ns[MN]	My[MNm]	Mz[MNm]	Vy[MN]	Vz[MN]	Mt[MNm]	z[m]
6903	min NS	-2170	2293	263	14	28	23	71
6903	max NS	-831	-3957	-37	7	-3	-38	71
6903	min MY	-1008	-7545	15	4	-56	21	71
6903	max MY	-1915	4778	-30	1	71	-35	71
6903	min MZ	-1809	1402	-541	-13	16	94	71
6903	max MZ	-1374	-2880	509	7	-18	-97	71
6903	min VY	-1749	225	-213	-32	7	114	71
6903	max VY	-1353	-1827	143	32	19	-135	71
6903	min L001	-2099	4338	174	10	40	-22	71
6903	max L001	-955	-7450	27	6	-46	24	71
6903	min L002	-2151	3814	292	14	37	-28	71
6903	max L002	-920	-7007	-147	-4	-30	9	71

Sectional forces from IBDAS beam model for Uniform Temperature Loading

Case	Criteria	Ns[MN]	My[MNm]	Mz[MNm]	Vy[MN]	Vz[MN]	Mt[MNm]	z[m]
4510	min NS	-3	-228	20	0	0	-1	71
4510	max NS	4	244	-36	-1	0	1	71
4510	min MY	-3	-228	20	0	0	-1	71
4510	max MY	4	244	-36	-1	0	1	71
4510	min MZ	4	244	-36	-1	0	1	71
4510	max MZ	-3	-228	20	0	0	-1	71
4510	min VY	4	244	-36	-1	0	1	71
4510	max VY	-3	-228	20	0	0	-1	71
4510	min L001	4	244	-36	-1	0	1	71
4510	max L001	-3	-228	20	0	0	-1	71
4510	min L002	4	244	-36	-1	0	1	71
4510	max L002	-3	-228	20	0	0	-1	71

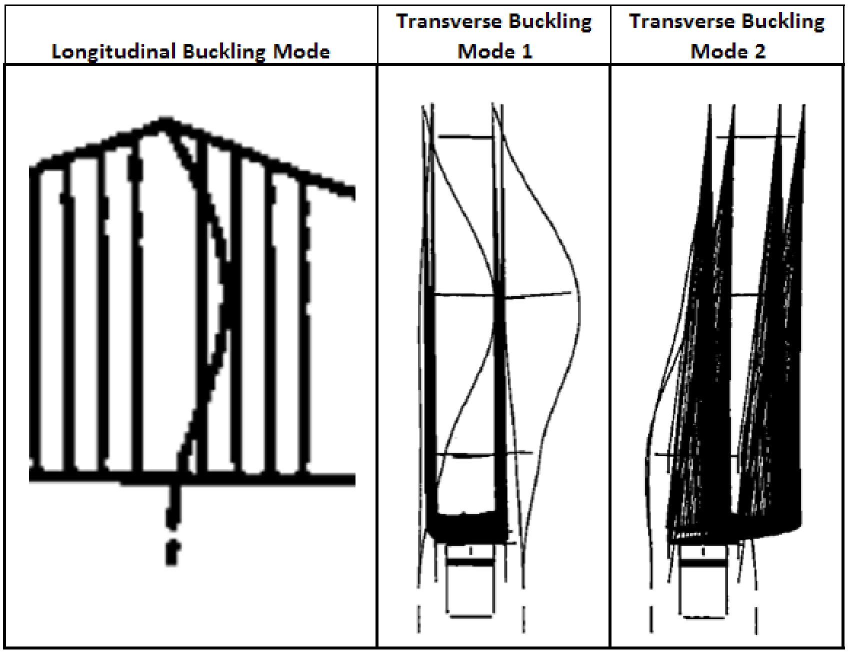
5.1.1.3 Global Stability of the Tower Legs

In addition to the sectional forces from the IBDAS global model, additional stresses accounting for the global buckling of the tower must be considered. The additional stresses are determined using equivalent imperfections as described in CG.10.00-P-RG-D-P-SV-T4-00-00-00-00-01 “General Design Principles.”

The deflected shape of the tower under various buckling modes is determined using an Eigenvalue buckling analysis. The equivalent imperfections imposed on the tower follow the same deflected shape.

The eigenvalue buckling analysis provides the critical elastic buckling force associated with each mode. The critical elastic buckling force is provided as a scaled value of the reference dead load in the eigenvalue buckling analysis.

The following table presents the three relevant buckling modes that were considered for the tower design and the associated dead load scaling factor for computing the critical elastic buckling force.

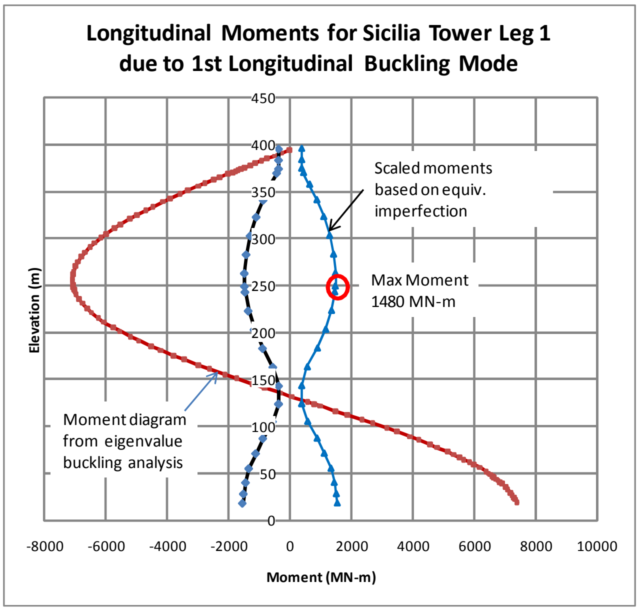


For the three buckling modes considered, the following table shows the critical elastic bucking load for the critical section as well as the calculated equivalent imperfection. The critical section was determined as the section with the highest stresses due to the buckling moments.

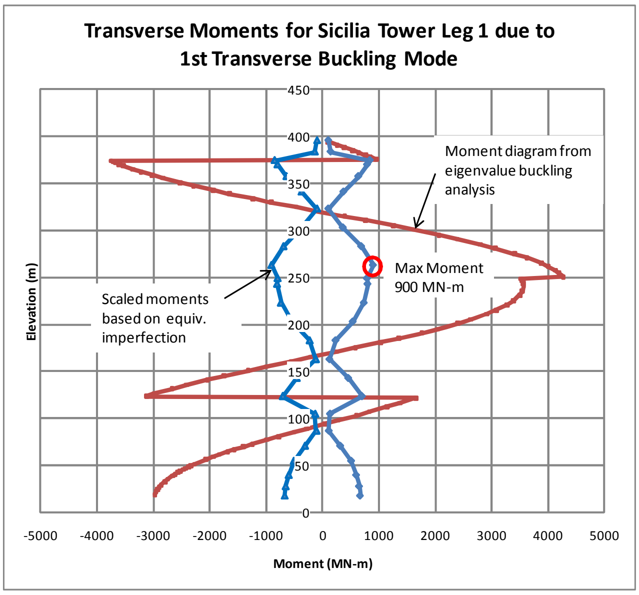
Buckling Mode	**Scalar value for critical elastic buckling load**	**Equivalent Imperfection, e_o (m)**	**Elastic Amplification for Moment**	**Imperfection Moment at Critical Section (MN-m)**
1st longitudinal	7.66	0.70	1.24	1480
1st transverse	6.72	0.44	1.28	900
2nd transverse	9.11	0.34	1.19	650

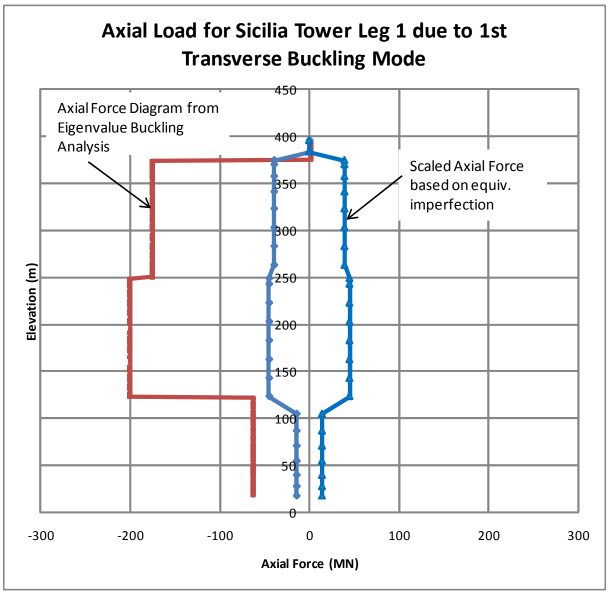
Using the above maximum buckling moments at the critical section, the moment diagrams from the eigenvalue buckling analysis were scaled as appropriate to achieve the same maximum bending moment at the critical section. The following diagrams show the bending moment diagrams from the eigenvalue bucking analysis as well as the resulting scaled moments to be used in the tower design.

1st longitudinal buckling mode

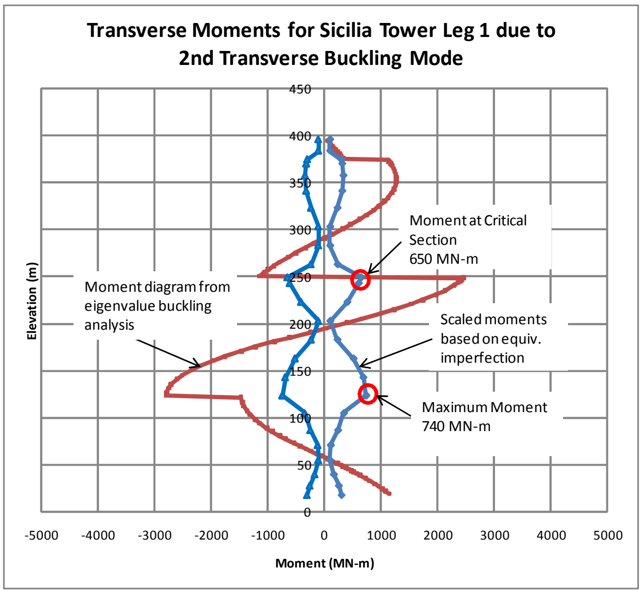


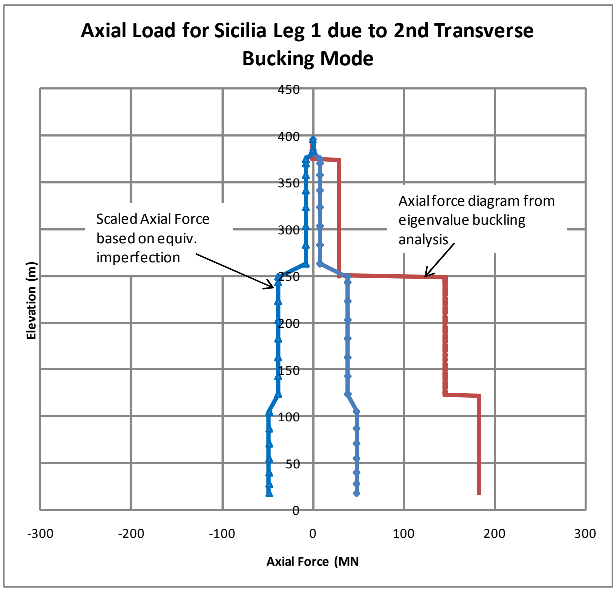
1st Transverse Buckling Mode





2nd Transverse Buckling Mode





From the force diagrams shown above, the section forces due to global buckling of the towers can be determined for each tower segment. For the tower segment 5 being considered, these forces are:

1st Longitudinal Buckling

Moment	1109	MN-m
Axial Load	0	MN

1st Transverse Buckling Mode

Moment	315	MN-m
Axial Load	14	MN

2nd Transverse Buckling Mode

Moment	114	MN-m
Axial Load	48	MN

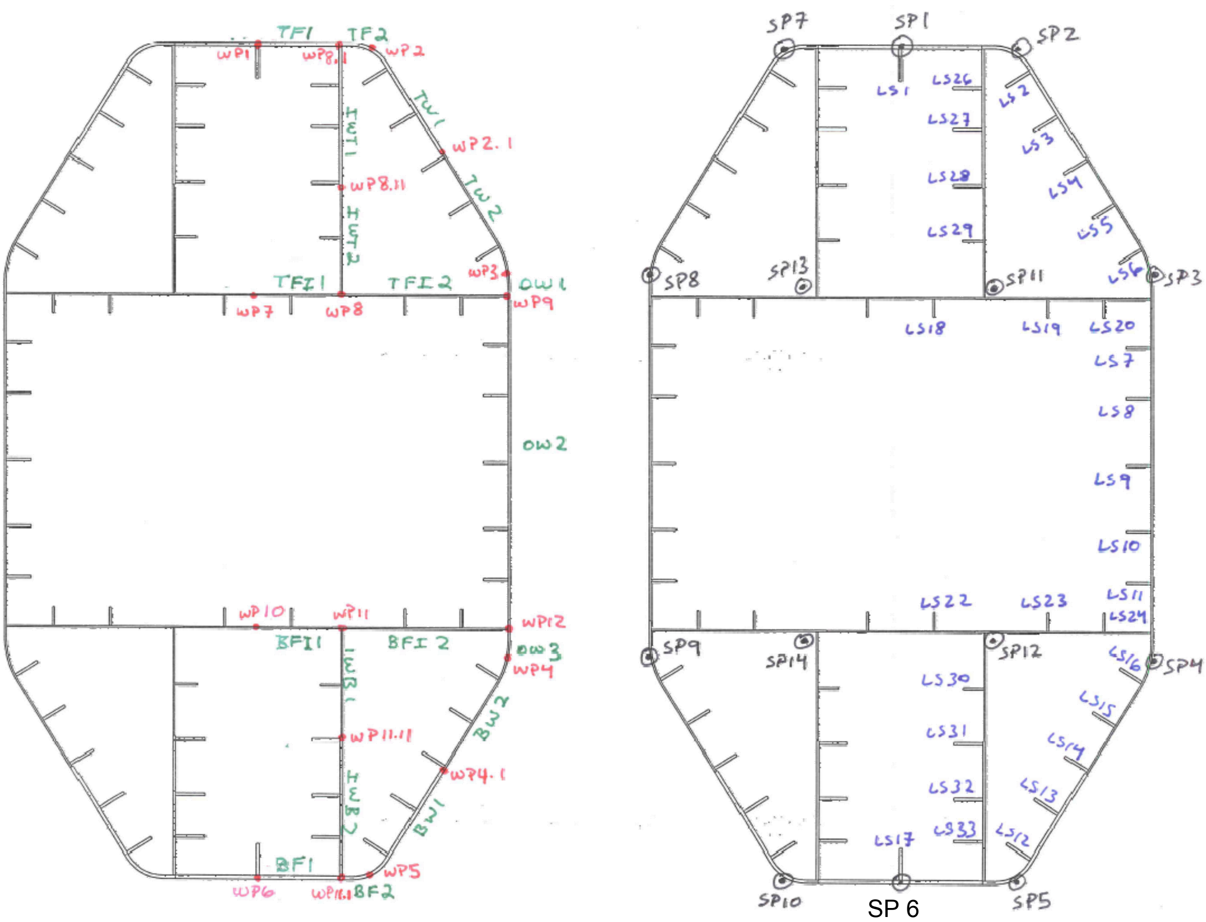
These section forces must be taken as plus/minus and are added to the section forces from the global model, shown previously. Only one buckling mode needs to be considered at a time; therefore, stresses are calculated for each buckling mode and maximum stress for each point on the cross section are used for the overall design.

5.1.1.4 Capacity of the Cross Section

As described in CG.10.00-P-RG-D-P-SV-T4-00-00-00-00-01 “General Design Principles,” the capacity of the tower cross section for longitudinal stresses was determined by calculating the buckling capacity of each longitudinal stiffener and unstiffened corner of the cross section. The buckling capacity of the longitudinal stiffeners is a based on reduction factors for both local buckling of the stiffener/main plates, column-like buckling of the stiffener and the associated tributary width of the main plate and plate-like buckling for the single longitudinal stiffeners on plate A (LS1 and LS17). The buckling capacity of the unstiffened corners includes only reductions for local plate buckling as the adjacent intersecting main plate will prevent any overall column-like buckling in these areas.

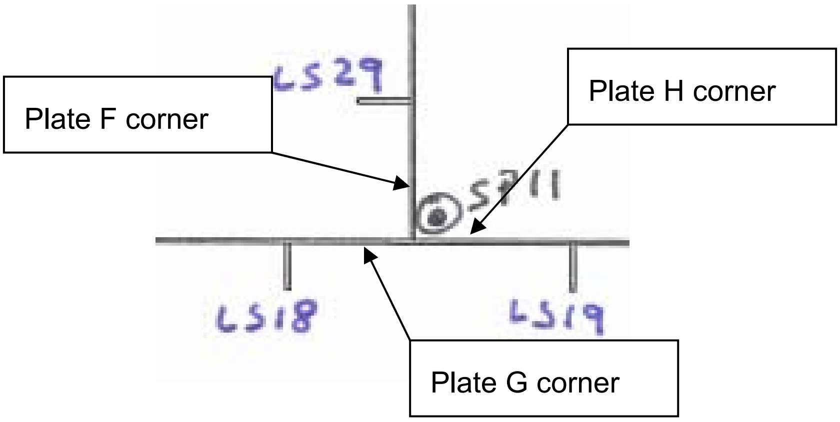
The following sketch shows the designations used for the cross-section in the spreadsheet calculations to follow.

The cross-section on the left details the work points and plate designations used to define the cross-section geometry. Shown on the right are designations for all longitudinal stiffeners and unstiffened corners in the cross section. Each longitudinal stiffener is assigned a unique name with the prefix “LS” and stresses for a given stiffener are conservatively calculated at the intersection of the stiffener and main plate. The unstiffened corner stress points are designated as “SP”.



In the reporting of the results, the tables include calculations for stiffeners "LS 21" and "LS 25". These stiffeners do not exist on the cross section and are used as "dummy" stiffeners to facilitate modifications to the cross sections. In the spreadsheet the location and capacities of the dummy stiffeners are arbitrarily taken equal to stiffeners "LS 20" and "LS 24" and therefore have no impact on the design of the cross section.

The unstiffened corner stress points account for the capacity of each plate type at a given panel intersection point. The stress points are conservatively placed at the critical stress location of the unstiffened corners that they are representing. For example, the capacity of “SP11” shown above is taken as the minimum capacity of the unstiffened corner for plates F, H and G.



Similarly, stress point “SP2” accounts for the unstiffened corners on plates A, B and E. Although this approach is slightly conservative, in reality it has minimal impact on the overall design, as the corners governing the cross section capacity are generally proportioned to be fully effective.

As the cross-section is symmetrical, only one-half of the cross-section needs to be specified, although stresses are calculated for all points on the cross-section. Therefore, each longitudinal stiffener will have two stress values, representing each half of the cross-section.

The column-like buckling capacities are not shown for stiffeners “LS1” and “LS17”, located on plate A. Due to the large thickness of plate A relative to the panel width and the fact that transverse stiffeners are not provided at regular intervals (panel length of up to 20 m), plate-like buckling dominates the behaviour and was accounted for using EN 1993-1-5 Annex A.2.2.. The capacity of plate A is represented by the capacity of stress point “SP1”. The calculations of the capacities for "LS1" and "LS17" are presented in later sections.

The following sections show the calculated buckling capacities of each longitudinal stiffener and unstiffened corner on the cross-section for Sicilia tower leg segment 5.

Capacity of Longitudinal Stiffeners:

Name			LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11
cross section type			0	0	0	0	0	0	0	0	0	0
No. in sec			2	2	2	2	2	2	2	2	2	2
Type			T1	T1	T1	T1	T1	T1	T1	T1	T1	T1
Plate #1	Reference	Unit										
b_{pl}		[mm]	1333	1250	1250	1250	1250	1200	1200	1600	1600	1200
t_{pl}		[mm]	90	90	45	45	45	40	40	40	40	40
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]	0.35	0.32	0.65	0.65	0.65	0.70	0.70	0.95	0.95	0.70
ρ_{pl}	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	0.98	0.98	0.81	0.81	0.98
$b_{pl,eff}$		[mm]	1333	1250	1250	1250	1250	1174	1174	1296	1296	1174
Plate #2												
b_{pl}		[mm]	1250	1250	1250	1250	1181	1200	1600	1600	1200	1200
t_{pl}		[mm]	90	90	45	45	45	40	40	40	40	40
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]	0.32	0.32	0.65	0.65	0.61	0.70	0.95	0.95	0.70	0.70
ρ_{pl}	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	0.98	0.81	0.81	0.98	0.98
$b_{pl,eff}$		[mm]	1250	1250	1250	1250	1181	1174	1296	1296	1174	1174
Total Plate												
tpl		[mm]	90	90	45	45	45	40	40	40	40	40
b_total gross		[mm]	1292	1250	1250	1250	1216	1200	1400	1600	1400	1200
b_total effec		[mm]	1292	1250	1250	1250	1216	1174	1235	1296	1235	1174
$\rho_{pl,\ composite}$			1.00	1.00	1.00	1.00	1.00	0.98	0.88	0.81	0.88	0.98
Stiffener web												
h_w		[mm]	675	675	625	625	625	600	600	600	600	600
t_w		[mm]	68	68	63	63	63	60	60	60	60	60
ψ_w	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,w}$	5: Table 4.1	[-]	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,w}$	5: (4.4)	[-]	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
ρ_w	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$h_{w,eff}$		[mm]	675.00	675.00	625.00	625.00	625.00	598.80	598.80	598.80	598.80	598.80
$t_{w,eff.\ equiv.}$		[mm]	68.00	68.00	63.00	63.00	63.00	59.88	59.88	59.88	59.88	59.88
Column Buckling of Longitudinal Stiffener												
L_{cr}		[mm]	3200	3200	3200	3200	3200	3200	3200	3200	3200	3200
α	1: Table 6.1	[-]	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49
Gross section												
A_c		[mm^2]	162135	158400	95625	95625	94073	84000	92000	100000	92000	84000
S		[mm^3]	2.49E+07	2.47E+07	1.53E+07	1.53E+07	1.53E+07	1.32E+07	1.34E+07	1.35E+07	1.34E+07	1.32E+07
y_o (from t/o plate)		[mm]	153	156	160	160	163	157	145	135	145	157
I_{xx}		[mm^4]	1.04E+10	1.04E+10	6.35E+09	6.35E+09	6.35E+09	5.27E+09	5.27E+09	5.28E+09	5.27E+09	5.27E+09
I		[mm^4]	6.636E+09	6.588E+09	3.891E+09	3.891E+09	3.860E+09	3.193E+09	3.331E+09	3.448E+09	3.331E+09	3.193E+09
Effective section												
$A_{c,eff}$		[mm^2]	162135	158400	95625	95625	94073	82875	85317	87759	85317	82875
S_{eff}		[mm^3]	2.4853E+07	2.4685E+07	1.5342E+07	1.5342E+07	1.5307E+07	1.3133E+07	1.3182E+07	1.3230E+07	1.3182E+07	1.3133E+07
$y_{o,eff}$ (from t/o plate)		[mm]	153.28	155.84	160.44	160.44	162.72	158.46	154.50	150.76	154.50	158.46
$I_{xx,eff}$		[mm^4]	1.045E+10	1.044E+10	6.352E+09	6.352E+09	6.351E+09	5.237E+09	5.238E+09	5.240E+09	5.238E+09	5.237E+09
I_{eff}		[mm^4]	6.636E+09	6.588E+09	3.891E+09	3.891E+09	3.860E+09	3.156E+09	3.202E+09	3.245E+09	3.202E+09	3.156E+09
Buckling												
eplate		mm	108	111	138	138	140	137	125	115	125	137
estiff		mm	274	272	197	197	195	183	195	205	195	183
emax	5: (4.5.3(5))	mm	274	272	197	197	195	183	195	205	195	183
i		mm	202	204	202	202	203	195	190	186	190	195
αe	5: (4.5.3(5))		0.612	0.610	0.578	0.578	0.577	0.574	0.582	0.589	0.582	0.574
$\sigma_{cr,sl}$	5: (4.5.3)	MPa	8284	8418	8235	8235	8306	7694	7329	6979	7329	7694
bc/bs1	5: (4.5.3)		1	1	1	1	1	1	1	1	1	1
$\sigma_{cr,c}$	5: (4.5.3)	MPa	8284	8418	8235	8235	8306	7694	7329	6979	7329	7694
βac	5: (4.5.3)		1.00	1.00	1.00	1.00	1.00	0.99	0.93	0.88	0.93	0.99
λc	5: (4.5.3)		0.236	0.234	0.236	0.236	0.235	0.243	0.241	0.241	0.241	0.243
Φ	1: (6.3.1.2)	[-]	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54	0.54
χ	1: (6.3.1.2)		0.98	0.98	0.98	0.98	0.98	0.97	0.98	0.98	0.98	0.97
Overall critical stress												
Min value			**409**	**409**	**409**	**409**	**409**	**402**	**378**	**358**	**378**	**402**
Total reduction			**0.9774**	**0.9787**	**0.9783**	**0.9783**	**0.9789**	**0.9615**	**0.9043**	**0.8559**	**0.9043**	**0.9615**

Name			LS12	LS13	LS14	LS15	LS16	LS17	LS18	LS19	LS20	LS21	LS22
cross section type			0	0	0	0	0	0	0	0	0	0	0
No. in sec			2	2	2	2	2	1	2	2	2	0	2
Type			T1	T1	T1	T1	T1	T1	T1	T1	T1	T1	T1
Plate #1	Reference	Unit											
b_{pl}		[mm]	1333	1250	1250	1250	1250	2000	1600	1500	1325	1325	1600
t_{pl}		[mm]	90	90	45	45	45	110	35	40	40	40	35
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4	4
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]	0.35	0.32	0.65	0.65	0.65	0.43	1.09	0.90	0.79	0.79	1.09
ρ_{pl}	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	1.00	0.73	0.84	0.91	0.91	0.73
$b_{pl,eff}$		[mm]	1333	1250	1250	1250	1250	2000	1168	1263	1212	1212	1168
Plate #2													
b_{pl}		[mm]	1250	1250	1250	1250	1181	2000	1200	1325	1175	1175	1200
t_{pl}		[mm]	90	90	45	45	45	110	35	40	40	40	35
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4	4
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]	0.32	0.32	0.65	0.65	0.61	0.43	0.81	0.79	0.70	0.70	0.81
ρ_{pl}	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	1.00	0.90	0.91	0.98	0.98	0.90
$b_{pl,eff}$		[mm]	1250	1250	1250	1250	1181	2000	1077	1212	1155	1155	1077
Total Plate													
tpl		[mm]	90	90	45	45	45	110	35	40	40	40	35
b_total gross		[mm]	1292	1250	1250	1250	1216	2000	1400	1413	1250	1250	1400
b_total effec		[mm]	1292	1250	1250	1250	1216	2000	1122	1237	1183	1183	1122
$\rho_{pl,\ composite}$			1.00	1.00	1.00	1.00	1.00	1.00	0.80	0.88	0.95	0.95	0.80
Stiffener web													
h_w		[mm]	675	675	625	625	625	750	450	450	450	450	450
t_w		[mm]	68	68	63	63	63	75	45	45	45	45	45
ψ_w	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,w}$	5: Table 4.1	[-]	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,w}$	5: (4.4)	[-]	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
ρ_w	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$h_{w,eff}$		[mm]	675.00	675.00	625.00	625.00	625.00	748.49	449.10	449.10	449.10	449.10	449.10
$t_{w,eff.\ equiv.}$		[mm]	68.00	68.00	63.00	63.00	63.00	74.85	44.91	44.91	44.91	44.91	44.91
Column Buckling of Longitudinal Stiffener													
L_{cr}		[mm]	3200	3200	3200	3200	3200	3200	3200	3200	3200	3200	3200
α	1: Table 6.1	[-]	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49
Gross section													
A_c		[mm^2]	162135	158400	95625	95625	94073	276250	69250	76750	70250	70250	69250
S		[mm^3]	2.49E+07	2.47E+07	1.53E+07	1.53E+07	1.53E+07	3.94E+07	6.12E+06	6.50E+06	6.37E+06	6.37E+06	6.12E+06
y_0 (from t/o plate)		[mm]	153	156	160	160	163	143	88	85	91	91	88
I_{xx}		[mm^4]	1.04E+10	1.04E+10	6.35E+09	6.35E+09	6.35E+09	1.68E+10	1.73E+09	1.79E+09	1.79E+09	1.79E+09	1.73E+09
I		[mm^4]	6.636E+09	6.588E+09	3.891E+09	3.891E+09	3.860E+09	1.114E+10	1.189E+09	1.244E+09	1.214E+09	1.214E+09	1.189E+09
Effective section													
$A_{c,eff}$		[mm^2]	162135	158400	95625	95625	94073	276137	59494	69707	67540	67540	59494
S_{eff}		[mm^3]	2.4853E+07	2.4685E+07	1.5342E+07	1.5342E+07	1.5307E+07	3.9284E+07	5.9328E+06	6.3363E+06	6.2930E+06	6.2930E+06	5.9328E+06
$y_{0,eff}$ (from t/o plate)		[mm]	153.28	155.84	160.44	160.44	162.72	142.26	99.72	90.90	93.17	93.17	99.72
$I_{xx,eff}$		[mm^4]	1.045E+10	1.044E+10	6.352E+09	6.352E+09	6.351E+09	1.667E+10	1.717E+09	1.780E+09	1.779E+09	1.779E+09	1.717E+09
I_{eff}		[mm^4]	6.636E+09	6.588E+09	3.891E+09	3.891E+09	3.860E+09	1.108E+10	1.125E+09	1.204E+09	1.193E+09	1.193E+09	1.125E+09
Buckling													
eplate		mm	108	111	138	138	140	88	71	65	71	71	71
estiff		mm	274	272	197	197	195	342	172	180	174	174	172
emax	5: (4.5.3(5))	mm	274	272	197	197	195	342	172	180	174	174	172
i		mm	202	204	202	202	203	201	131	127	131	131	131
αe	5: (4.5.3(5))		0.612	0.610	0.578	0.578	0.577	0.643	0.608	0.617	0.609	0.609	0.608
$\sigma_{cr,sl}$	5: (4.5.3)	MPa	8284	8418	8235	8235	8306	8163	3476	3281	3496	3496	3476
bc/bs1	5: (4.5.3)		1	1	1	1	1	1	1	1	1	1	1
$\sigma_{cr,c}$	5: (4.5.3)	MPa	8284	8418	8235	8235	8306	8163	3476	3281	3496	3496	3476
βac	5: (4.5.3)		1.00	1.00	1.00	1.00	1.00	1.00	0.86	0.91	0.96	0.96	0.86
λc	5: (4.5.3)		0.236	0.234	0.236	0.236	0.235	0.237	0.337	0.357	0.356	0.356	0.337
Φ	1: (6.3.1.2)	[-]	0.54	0.54	0.54	0.54	0.54	0.54	0.60	0.61	0.61	0.61	0.60
χ	1: (6.3.1.2)		0.98	0.98	0.98	0.98	0.98	0.98	0.91	0.90	0.90	0.90	0.91
Overall critical stress													
Min value			**409**	**409**	**409**	**409**	**409**	**408**	**329**	**342**	**363**	**363**	**329**
Total reduction			**0.9774**	**0.9787**	**0.9783**	**0.9783**	**0.9789**	**0.9748**	**0.7860**	**0.8187**	**0.8684**	**0.8684**	**0.7860**

Name			LS23	LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
cross section type			0	0	0	0	0	0	0	0	0	0	0
No. in sec			2	2	0	2	2	2	2	2	2	2	2
Type			T1	T1	T1	T1	T1	T1	T1	T1	T1	T1	T1
Plate #1	Reference	Unit											
b_{pl}		[mm]	1500	1325	1325	1000	1000	1333	1333	1333	1333	1000	1000
t_{pl}		[mm]	40	40	40	60	60	60	35	35	60	60	60
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4	4
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]	0.90	0.79	0.79	0.38	0.38	0.52	0.90	0.90	0.52	0.38	0.38
ρ_{pl}	5: (4.4)	[-]	0.84	0.91	0.91	1.00	1.00	1.00	0.84	0.84	1.00	1.00	1.00
$b_{pl,eff}$		[mm]	1263	1212	1212	1000	1000	1333	1117	1117	1333	1000	1000
Plate #2													
b_{pl}		[mm]	1325	1175	1175	1000	1333	1333	1333	1333	1333	1333	1000
t_{pl}		[mm]	40	40	40	60	60	60	35	35	60	60	60
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4	4
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]	0.79	0.70	0.70	0.38	0.52	0.52	0.90	0.90	0.52	0.52	0.38
ρ_{pl}	5: (4.4)	[-]	0.91	0.98	0.98	1.00	1.00	1.00	0.84	0.84	1.00	1.00	1.00
$b_{pl,eff}$		[mm]	1212	1155	1155	1000	1333	1333	1117	1117	1333	1333	1000
Total Plate													
tpl		[mm]	40	40	40	60	60	60	35	35	60	60	60
b_total gross		[mm]	1413	1250	1250	1000	1167	1333	1333	1333	1333	1167	1000
b_total effec		[mm]	1237	1183	1183	1000	1167	1333	1117	1117	1333	1167	1000
$\rho_{pl,\ composite}$			0.88	0.95	0.95	1.00	1.00	1.00	0.84	0.84	1.00	1.00	1.00
Stiffener web													
h_w		[mm]	450	450	450	675	675	675	500	500	675	675	675
t_w		[mm]	45	45	45	68	68	68	50	50	68	68	68
ψ_w	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,w}$	5: Table 4.1	[-]	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43	0.43
f_y		[MPa]	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,w}$	5: (4.4)	[-]	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75	0.75
ρ_w	5: (4.4)	[-]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$h_{w,eff}$		[mm]	449.10	449.10	449.10	675.00	675.00	675.00	499.00	499.00	675.00	675.00	675.00
$t_{w,eff.\ equiv.}$		[mm]	44.91	44.91	44.91	68.00	68.00	68.00	49.90	49.90	68.00	68.00	68.00
Column Buckling of Longitudinal Stiffener													
L_{cr}		[mm]	3200	3200	3200	3200	3200	3200	3200	3200	3200	3200	3200
α	1: Table 6.1	[-]	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49	0.49
Gross section													
A_c		[mm^2]	76750	70250	70250	105900	115890	125880	71655	71655	125880	115890	105900
S		[mm^3]	6.50E+06	6.37E+06	6.37E+06	2.00E+07	2.03E+07	2.06E+07	7.94E+06	7.94E+06	2.06E+07	2.03E+07	2.00E+07
y_o (from t/o plate)		[mm]	85	91	91	189	176	164	111	111	164	176	189
I_{xx}		[mm^4]	1.79E+09	1.79E+09	1.79E+09	9.07E+09	9.08E+09	9.09E+09	2.57E+09	2.57E+09	9.09E+09	9.08E+09	9.07E+09
I		[mm^4]	1.244E+09	1.214E+09	1.214E+09	5.273E+09	5.508E+09	5.705E+09	1.690E+09	1.690E+09	5.705E+09	5.508E+09	5.273E+09
Effective section													
$A_{c,eff}$		[mm^2]	69707	67540	67540	105900	115890	125880	64031	64031	125880	115890	105900
S_{eff}		[mm^3]	6.3363E+06	6.2930E+06	6.2930E+06	2.0045E+07	2.0345E+07	2.0645E+07	7.7821E+06	7.7821E+06	2.0645E+07	2.0345E+07	2.0045E+07
$y_{o,eff}$ (from t/o plate)		[mm]	90.90	93.17	93.17	189.28	175.55	164.00	121.54	121.54	164.00	175.55	189.28
$I_{xx,eff}$		[mm^4]	1.780E+09	1.779E+09	1.779E+09	9.067E+09	9.079E+09	9.091E+09	2.553E+09	2.553E+09	9.091E+09	9.079E+09	9.067E+09
I_{eff}		[mm^4]	1.204E+09	1.193E+09	1.193E+09	5.273E+09	5.508E+09	5.705E+09	1.607E+09	1.607E+09	5.705E+09	5.508E+09	5.273E+09
Buckling													
eplate		mm	65	71	71	159	146	134	93	93	134	146	159
estiff		mm	180	174	174	208	222	233	174	174	233	222	208
emax	5: (4.5.3(5))	mm	180	174	174	208	222	233	174	174	233	222	208
i		mm	127	131	131	223	218	213	154	154	213	218	223
αe	5: (4.5.3(5))		0.617	0.609	0.609	0.574	0.582	0.589	0.592	0.592	0.589	0.582	0.574
$\sigma_{cr,sl}$	5: (4.5.3)	MPa	3281	3496	3496	10078	9619	9174	4775	4775	9174	9619	10078
bc/bs1	5: (4.5.3)		1	1	1	1	1	1	1	1	1	2	3
$\sigma_{cr,c}$	5: (4.5.3)	MPa	3281	3496	3496	10078	9619	9174	4775	4775	9174	19238	30234
βac	5: (4.5.3)		0.91	0.96	0.96	1.00	1.00	1.00	0.89	0.89	1.00	1.00	1.00
λc	5: (4.5.3)		0.357	0.356	0.356	0.214	0.219	0.224	0.293	0.293	0.224	0.155	0.123
Φ	1: (6.3.1.2)	[-]	0.61	0.61	0.61	0.53	0.53	0.53	0.57	0.57	0.53	0.50	0.49
χ	1: (6.3.1.2)		0.90	0.90	0.90	0.99	0.99	0.99	0.94	0.94	0.99	1.00	1.00
Overall critical stress													
Min value			**342**	**363**	**363**	**415**	**413**	**412**	**352**	**352**	**412**	**418**	**418**
Total reduction			**0.8187**	**0.8684**	**0.8684**	**0.9919**	**0.9887**	**0.9854**	**0.8429**	**0.8429**	**0.9854**	**1.0000**	**1.0000**

Capacity of Plate A Stiffeners "LS1" and "LS17":

As discussed previously, the capacity of the stiffeners located on Plate A were calculated accounting for both column-like and plate-like buckling. The plate-like buckling capacity was determined using EN 1993-1-5 Annex A.2.2. The reduction factors are based conservatively on the 20 m maximum distance between transverse stiffeners/diaphragms on Plate A. The following table presents the effectiveness of these stiffeners for the range of plate A thicknesses.

Plate Thickness (mm)	Overall Redcution Factor	Max Critical Stress (Mpa)
55	0.86	360
60	0.91	381
65	0.95	397
70	0.98	410
75	1	418

From the table it can be seen that Plate A and its stiffeners are fully effective for plate thicknesses greater than 75mm, and so detailed calculations are not presented for these thicker plates. The capacities for Plate A are used in checking stress at "SP1".

The following are the detailed calculations used to determine the reduction factors for Plate A stiffeners shown in the above table.

Calculation of Plate A and LS1 Capacity

Assume conservatively that the entire panel is uniformly compressed and thus: $\Psi := 1.0$

EN 1993-1-5 Section 4.5: Stiffened plate elements with longitudinal stiffeners

LS1 is centred on plate A, so: $b_1 := 2 \cdot m$ and $b_2 := 2 \cdot m$

Determine the effective widths of plate A and LS1 using Section 4.4:

Thickness of plate A: $t_A := (55 \;\; 60 \;\; 65 \;\; 70 \;\; 75)^T \cdot mm$ $\qquad i := 1 .. 5$

Steel yield strength: $f_y := 460 \cdot MPa$

$$\varepsilon := \sqrt{\frac{235 \cdot MPa}{f_y}} \qquad \varepsilon = 0.715$$

Buckling factor: $k_\sigma := 4$

Plate A:

Plate slenderness:

$$\lambda_{p_i} := \frac{\dfrac{b_1}{t_{A_i}}}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} \qquad \lambda_p{}^T = (0.896 \;\; 0.821 \;\; 0.758 \;\; 0.704 \;\; 0.657)$$

Reduction factors:

$$\rho_i := \begin{cases} 1 & \text{if } \lambda_{p_i} \le 0.673 \\ \min\left[\dfrac{\lambda_{p_i} - 0.055 \cdot (3 + \Psi)}{\left(\lambda_{p_i}\right)^2}, 1\right] & \text{if } \lambda_{p_i} > 0.673 \end{cases}$$

$$\rho^T = (0.842 \;\; 0.892 \;\; 0.936 \;\; 0.977 \;\; 1)$$

Longitudinal Stiffener LS1:

Buckling factor: $k_\sigma := 0.43$

LS1 dimensions: $b_{LS1} := 750 \cdot mm$ and $t_{LS1} := 75 \cdot mm$

Plate slenderness:

$$\lambda_{p\_LS1} := \frac{\dfrac{b_{LS1}}{t_{LS1}}}{28.4 \cdot \varepsilon \cdot \sqrt{k_\sigma}} \qquad \lambda_{p\_LS1} = 0.751$$

Reduction factors:

$$\rho_{LS1} := \begin{cases} 1 & \text{if } \lambda_{p\_LS1} \le 0.748 \\ \min\left(\dfrac{\lambda_{p\_LS1} - 0.188}{\lambda_{p\_LS1}^{2}}, 1\right) & \text{if } \lambda_{p\_LS1} > 0.748 \end{cases} \qquad \rho_{LS1} = 0.998$$

Effective area of longitudinal stiffener and plate A:

$$A_{c\_eff_i} := \rho_i \cdot 0.5 \cdot (b_1 + b_2) \cdot t_{A_i} + \rho_{LS1} \cdot b_{LS1} \cdot t_{LS1} \qquad A_{c\_eff}^{T} = (0.149 \;\; 0.163 \;\; 0.178 \;\; 0.193 \;\; 0.206)\, m^2$$

$$\rho_{loc_i} := \frac{A_{c\_eff_i}}{0.5 \cdot (b_1 + b_2) \cdot t_{A_i} + b_{LS1} \cdot t_{LS1}} \qquad \rho_{loc}^{T} = (0.895 \;\; 0.926 \;\; 0.955 \;\; 0.983 \;\; 0.999)$$

EN 1993-1-5 Section 4.5.2: Plate-type behaviour

Elastic critical plate buckling stress is calculated in accordance with Annex A, or more specifically Annex A.2.2 for plates with a single longitudinal stiffener.

Compute the gross properties of the single stiffener and tributary main plate in accordance with EN 1993-1-5 Annex A.2.1.

Tributary widths:

$$b_{1t} := \left(\frac{3 - \Psi}{5 - \Psi}\right) \cdot b_1 \qquad b_{1t} = 1\, m$$

$$b_{2t} := \left(\frac{2}{5 - \Psi}\right) \cdot b_2 \qquad b_{2t} = 1\, m$$

Area of stiffener and tributary plate A:

$$A_{sl1_i} := t_{A_i} \cdot (b_{1t} + b_{2t}) + b_{LS1} \cdot t_{LS1}$$

$$A_{sl1}^{T} = (0.166 \;\; 0.176 \;\; 0.186 \;\; 0.196 \;\; 0.206)\, m^2$$

Centroid of the stiffener and tributary plate A relative to the outside of plate A:

$$y_{bar_i} := \frac{0.5 \cdot \left(t_{A_i}\right)^2 \cdot (b_{1t} + b_{2t}) + b_{LS1} \cdot t_{LS1} \cdot \left(t_{A_i} + 0.5 \cdot b_{LS1}\right)}{A_{sl1_i}}$$

$$y_{bar}^{T} = (0.164 \;\; 0.159 \;\; 0.156 \;\; 0.153 \;\; 0.15)\, m$$

Moment of inertia of the stiffener and tributary plate A about the composite centroid:

$$I_{sl1_i} := \frac{\left(t_{A_i}\right)^3}{12} \cdot (b_{1t} + b_{2t}) + \frac{b_{LS1}^{3}}{12} \cdot t_{LS1} + t_{A_i} \cdot (b_{1t} + b_{2t}) \cdot \left(y_{bar_i} - 0.5 \cdot t_{A_i}\right)^2 \dots$$
$$+ b_{LS1} \cdot t_{LS1} \cdot \left[y_{bar_i} - \left(t_{A_i} + 0.5 \cdot b_{LS1}\right)\right]^2$$

$$I_{sl1}^{T} = \left(8.694\times10^{-3}\quad 8.955\times10^{-3}\quad 9.202\times10^{-3}\quad 9.439\times10^{-3}\quad 9.668\times10^{-3}\right)m^4$$

Panel length (distance between transverse stiffeners): $a := 20\cdot m$

Factor a_c: $a_{c_i} := 4.33\cdot\left[\frac{I_{sl1_i}\cdot b_1^2\cdot b_2^2}{\left(t_{A_i}\right)^3\cdot\left(b_1+b_2\right)}\right]^{0.25}$ $\quad a_c^{T} = (16.464\quad 15.538\quad 14.733\quad 14.025\quad 13.398)\,m$

Calculate the equivalent orthotropic plate buckling critical stress:

Elastic modulus: $E := 210000\cdot MPa$

Poisson's ratio: $\nu := 0.3$

$$\sigma_{cr\_sl_i} := \begin{cases} \dfrac{1.05\cdot E}{A_{sl1_i}}\cdot\dfrac{\sqrt{I_{sl1_i}\cdot\left(t_{A_i}\right)^3\cdot\left(b_1+b_2\right)}}{b_1\cdot b_2} & \text{if } a \ge a_{c_i} \\ \dfrac{\pi^2\cdot E\cdot I_{sl1_i}}{A_{sl1_i}\cdot a^2} + \dfrac{E\cdot\left(t_{A_i}\right)^3\cdot\left(b_1+b_2\right)\cdot a^2}{4\cdot\pi^2\cdot\left(1-\nu^2\right)\cdot A_{sl1_i}\cdot b_1^2\cdot b_2^2} & \text{if } a < a_{c_i} \end{cases}$$

$$\sigma_{cr\_sl}^{T} = (798\quad 870\quad 941\quad 1011\quad 1080)\,MPa$$

$$\sigma_p := \sigma_{cr\_sl}$$

Return to Section 4.5.2 with these elastic critical plate buckling stresses.

Effective to gross area ratios: $\beta_{AC_i} := \frac{A_{c\_eff_i}}{A_{sl1_i}}$ $\quad \beta_{AC}^{T} = (0.895\quad 0.926\quad 0.955\quad 0.983\quad 0.999)$

Effcetive plate slendernesses: $\lambda_{pp_i} := \sqrt{\frac{\beta_{AC_i}\cdot f_y}{\sigma_{p_i}}}$ $\quad \lambda_{pp}^{T} = (0.718\quad 0.7\quad 0.683\quad 0.669\quad 0.653)$

Reduction factor for plate type behaviour:

$$\rho_{p_i} := \begin{cases} 1 & \text{if } \lambda_{pp_i} \le 0.673 \\ \min\left[\dfrac{\lambda_{pp_i} - 0.055\cdot(3+\Psi)}{\left(\lambda_{pp_i}\right)^2}, 1\right] & \text{if } \lambda_{pp_i} > 0.673 \end{cases}$$

$$\rho_p^T = (0.966 \quad 0.98 \quad 0.992 \quad 1 \quad 1)$$

EN 1993-1-5 Section 4.5.3: Column-type behaviour

Critical elastic column buckling stress:

$$\sigma_{cr\_sl_i} := \frac{\pi^2 \cdot E \cdot I_{sl1_i}}{A_{sl1_i} \cdot a^2} \qquad \sigma_{cr\_sl}^T = (271 \quad 263.3 \quad 256 \quad 249.2 \quad 242.9)\,\mathrm{MPa}$$

Relative column slenderness:

In this case β_{AC} is the same as for plate-type behaviour.

$$\lambda_{c_i} := \sqrt{\frac{\beta_{AC_i} \cdot f_y}{\sigma_{cr\_sl_i}}} \qquad \lambda_c^T = (1.233 \quad 1.272 \quad 1.31 \quad 1.347 \quad 1.376)$$

Effective imperfection factor: $\alpha := 0.49$ for an open stiffener

$$r_i := \sqrt{\frac{I_{sl1_i}}{A_{sl1_i}}} \qquad r^T = (0.229 \quad 0.225 \quad 0.222 \quad 0.219 \quad 0.217)\,\mathrm{m}$$

radius of gyration, represented by i in EN 1993-1-5

$$e_i := \max\left[\left|y_{bar_i} - 0.5 \cdot t_{A_i}\right|, \left|y_{bar_i} - \left(t_{A_i} + 0.5 \cdot b_{LS1}\right)\right|\right]$$

$$e^T = (0.266 \quad 0.276 \quad 0.284 \quad 0.292 \quad 0.3)\,\mathrm{m}$$

$$\alpha_{e_i} := \alpha + \frac{0.09}{\dfrac{r_i}{e_i}} \qquad \alpha_e^T = (0.595 \quad 0.6 \quad 0.605 \quad 0.61 \quad 0.615)$$

Column-type behaviour reduction factor:

$$\Phi_i := 0.5\cdot\left[1 + \alpha_{e_i}\cdot\left(\lambda_{c_i} - 0.2\right) + \left(\lambda_{c_i}\right)^2\right] \qquad \Phi^T = (1.567\ \ 1.63\ \ 1.694\ \ 1.757\ \ 1.808)$$

$$\chi_{c_i} := \min\left[\frac{1}{\Phi_i + \sqrt{\left(\Phi_i\right)^2 - \left(\lambda_{c_i}\right)^2}}, 1\right] \qquad \chi_c^T = (0.395\ \ 0.377\ \ 0.361\ \ 0.347\ \ 0.336)$$

EN 1993-1-5 Section 4.5.4: Interaction between plate and column buckling

Interaction between column buckling and plate buckling:

$$\xi_i := \begin{cases} 0 & \text{if } \dfrac{\sigma_{p_i}}{\sigma_{cr\_sl_i}} - 1 < 0 \\ 1 & \text{if } \dfrac{\sigma_{p_i}}{\sigma_{cr\_sl_i}} - 1 > 1 \\ \dfrac{\sigma_{p_i}}{\sigma_{cr\_sl_i}} - 1 & \text{if } 0 \le \dfrac{\sigma_{p_i}}{\sigma_{cr\_sl_i}} - 1 \le 1 \end{cases} \qquad \xi^T = (1\ \ 1\ \ 1\ \ 1\ \ 1)$$

$\dfrac{\sigma_{p_i}}{\sigma_{cr\_sl_i}} =$

2.943
3.305
3.676
4.056
4.445

Final reduction factor: $\rho_{c_i} := \left(\rho_{p_i} - \chi_{c_i}\right)\cdot\xi_i\cdot\left(2 - \xi_i\right) + \chi_{c_i}$ $\qquad \rho_c^T = (0.966\ \ 0.98\ \ 0.992\ \ 1\ \ 1)$

Effective areas considering local plate element behaviour and overall buckling (plate and column) behaviour:

$$A_{eff_i} := \rho_{c_i}\cdot A_{c\_eff_i} \qquad A_{eff}^T = (0.144\ \ 0.16\ \ 0.177\ \ 0.193\ \ 0.206)\,m^2$$

Fraction of the yield stress that can be carried:

$$\rho_{eff_i} := \frac{A_{eff_i}}{t_{A_i}\cdot 0.5\cdot\left(b_1 + b_2\right) + b_{LS1}\cdot t_{LS1}} \qquad \rho_{eff}^T = (0.864\ \ 0.907\ \ 0.948\ \ 0.983\ \ 0.999)$$

Capacity of Unstiffened Corners:

The following are the capacities of the corner stress points on the cross section. It is important to note the stress point "SP1" represents the capacity of the Plate A stiffeners, for which details were given in the previous section

Segment	5
Analysis Secti	9
Elevation	71

PLATE A													
SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10	SP11	SP12	SP13	SP14
0	2640	5980.936	5980.936	2640	0	-2640	-5980.94	-5980.94	-2640	2412.5	2412.5	-2412.5	-2412.5
9945	9955	4628.049	-4628.05	-9955	-9945	9955	4628.049	-4628.05	-9955	4380	-4380	4380	-4380

mkleymann:
Represents Capacity of entire Panel A including interaction of plate and column buckling

Panel #1

Plate			TF1	TF1	OW2	OW2	BF1	BF1	TF1	OW2	OW2	BF1	TFI2	BFI2	TFI2	BFI2
Corner Spacing				2000	1200	1200	2000		2000	1200	1200	2000	1500	1500	1500	1500
plate thickness			110	110	40	40	110	110	110	40	40	110	40	40	40	40
$\psi_{,pl}$	5: Table 4.1	[-]		1	1	1	1		1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]		4	4	4	4		4	4	4	4	4	4	4	4
f_y	1: Table 3.1	[MPa]	460	460	460	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]		0.71	0.71	0.71	0.71		0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]		0.45	0.74	0.74	0.45		0.45	0.74	0.74	0.45	0.91	0.92	0.92	0.92
ρ_{pl}	5: (4.4)	[-]	1.00	1.00	0.95	0.95	1.00	1.00	1.00	0.95	0.95	1.00	0.83	0.82	0.82	0.82
max stress			418	418	397	397	418	418	418	397	397	418	348	345	345	345

Panel #2

Plate				IWT1	TFI2	BFI2	IWB2		IWT1	TFI2	BFI2	IWB2	TFI1	BFI1	TFI1	BFI1
Corner Spacing				1000	1175	1175	1000		1000	1175	1175	1000	1200	1200	1200	1200
plate thickness				60	40	40	60		60	40	40	60	35	35	35	35
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4	4	4	4	4
f_y	1: Table 3.1	[MPa]	460	460	460	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.4 (2))	[-]		0.71	0.71	0.71	0.71		0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.4)	[-]		0.41	0.72	0.72	0.41		0.41	0.72	0.72	0.41	0.81	0.81	0.84	0.84
ρ_{pl}	5: (4.4)	[-]		1.00	1.00	1.00	1.00		1.00	1.00	1.00	1.00	0.90	0.90	0.88	0.88
max stress			418	418	418	418	418	418	418	418	418	418	376	376	366	366

Panel #3

Plate				TW1	TW2	BW2	BW1		TW1	TW2	BW2	BW1	IWT2	IWB1	IWT2	IWB1
Corner Spacing				1333	1181	1181	1333		1333	1181	1181	1333	1333	1333	1333	1333
plate thickness				90	45	45	90		90	45	45	90	35	35	35	35
$\psi_{,pl}$	5: Table 4.1	[-]	1	1	1	1	1	1	1	1	1	1	1	1	1	1
$K_{\sigma,pl}$	5: Table 4.1	[-]	4	4	4	4	4	4	4	4	4	4	4	4	4	4
f_y	1: Table 3.1	[MPa]	460	460	460	460	460	460	460	460	460	460	460	460	460	460
ε	5: (4.3)	[-]		0.71	0.71	0.71	0.71		0.71	0.71	0.71	0.71	0.71	0.71	0.71	0.71
$\lambda_{p,pl}$	5: (4.3)	[-]		0.36	0.65	0.65	0.36		0.36	0.65	0.65	0.36	0.92	0.92	0.94	0.94
ρ_{pl}	5: (4.2)	[-]		1.00	1.00	1.00	1.00		1.00	1.00	1.00	1.00	0.82	0.82	0.82	0.82
max stress			418	418	418	418	418	418	418	418	418	418	345	345	341	341

	SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10	SP11	SP12	SP13	SP14
Governing design Stress	418	418	397	397	418	418	418	397	397	418	345	345	341	341
Governing reduction factor	1.00	1.00	0.95	0.95	1.00	1.00	1.00	0.95	0.95	1.00	0.82	0.82	0.82	0.82

5.1.1.5 Calculation of Stresses and Utilization Ratios

The section forces along with the section properties shown previously were used to calculate the maximum stresses for each stress point on the cross-section. The stress points correspond to the longitudinal stiffeners and corners, for which the capacities were shown in the previous section.

The following tables show the stress calculation for each stress point and the resulting utilization ratio using the capacities. All stresses and utilization ratios are for Sicilia tower segment 5 under load combination ULS 7, for which section forces were previously shown.

			PLATE STRESS POINTS													
Stress point			SP1	SP2	SP3	SP4	SP5	SP6	SP7	SP8	SP9	SP10	SP11	SP12	SP13	SP14
		side of cross section														
Plate ID			TF1	TF2	TW2	BW2	BF2	BF1	TF2	TW2	BW2	BF2	TFI2	BFI2	TFI2	BFI2
Location of	Y	(mm)	0	2640	5980.936	5980.936	2640	0	-2640	-5980.936	-5980.94	-2640	2412.5	2412.5	-2412.5	-2412.5
Stress point	Z	(mm)	9945	9955	4628.049	-4628.049	-9955	-9945	9955	4628.049	-4628.049	-9955	4380	-4380	4380	-4380
IBDAS ULS/SILS SECTION FORCES	σ_1 (Mpa)	min NS	-334	-341	-318	-265	-226	-219	-327	-288	-235	-213	-308	-258	-296	-245
		max NS	-7	-6	-58	-150	-204	-205	-8	-62	-154	-206	-62	-149	-63	-150
		min MY	60	60	-42	-217	-318	-317	60	-40	-215	-317	-46	-212	-45	-211
		max MY	-363	-363	-298	-187	-124	-125	-364	-301	-190	-126	-296	-191	-297	-192
		min MZ	-266	-252	-216	-183	-182	-196	-279	-278	-245	-209	-234	-203	-259	-228
		max MZ	-103	-116	-171	-238	-260	-247	-90	-113	-179	-234	-155	-219	-132	-195
		min VY	-229	-223	-214	-208	-212	-217	-234	-238	-233	-223	-221	-216	-231	-226
		max VY	-127	-130	-159	-202	-222	-218	-123	-143	-186	-215	-156	-196	-149	-189
		min L001	-376	-381	-328	-227	-164	-159	-372	-308	-207	-155	-319	-224	-311	-216
		max L001	64	64	-37	-210	-309	-308	65	-34	-207	-307	-41	-204	-39	-203
		min L002	-370	-377	-335	-247	-186	-179	-362	-302	-213	-172	-323	-239	-310	-226
		max L002	58	62	-28	-190	-289	-292	54	-44	-207	-296	-37	-191	-44	-198
		MIN	-376	-381	-335	-265	-318	-317	-372	-308	-245	-317	-323	-258	-311	-245
		MAX	64	64	-28	-150	-124	-125	65	-34	-154	-126	-37	-149	-39	-150
			min L001	*min L001*	*min L002*	*min NS*	*min MY*	*min MY*	*min L001*	*min L001*	*min MZ*	*min MY*	*min L002*	*min NS*	*min L001*	*min NS*
TEMP. LOAD CASE 4500	σ_1 (Mpa)	min NS	5	5	1	-4	-7	-6	6	3	-2	-6	2	-3	3	-2
		max NS	-6	-5	0	5	8	7	-7	-4	1	6	-1	4	-3	2
		min MY	5	5	1	-4	-7	-6	6	3	-2	-6	2	-3	3	-2
		max MY	-6	-5	0	5	8	7	-7	-4	1	6	-1	4	-3	2
		min MZ	-6	-5	0	5	8	7	-7	-4	1	6	-1	4	-3	2
		max MZ	5	5	1	-4	-7	-6	6	3	-2	-6	2	-3	3	-2
		min VY	-6	-5	0	5	8	7	-7	-4	1	6	-1	4	-3	2
		max VY	5	5	1	-4	-7	-6	6	3	-2	-6	2	-3	3	-2
		min L001	-6	-5	0	5	8	7	-7	-4	1	6	-1	4	-3	2
		max L001	5	5	1	-4	-7	-6	6	3	-2	-6	2	-3	3	-2
		min L002	-6	-5	0	5	8	7	-7	-4	1	6	-1	4	-3	2
		max L002	5	5	1	-4	-7	-6	6	3	-2	-6	2	-3	3	-2
		MIN	-6	-5	0	-4	-7	-6	-7	-4	-2	-6	-1	-3	-3	-2
		MAX	5	5	1	5	8	7	6	3	1	6	2	4	3	2
ULS/SILS + Temp		MIN	-382	-385	-336	-269	-324	-323	-378	-313	-247	-322	-324	-261	-314	-248
		MAX	69	68	-26	-144	-117	-118	71	-30	-153	-120	-35	-145	-37	-148
IMPERFECTION STRESSES																
1st Longitudinal Buckling																
Moment	1109	MN-m														
Axial Load	0.0	MN														
	σ_1	Max	27.70	27.72	12.89	-12.89	-27.72	-27.70	27.72	12.89	-12.89	-27.72	12.20	-12.20	12.20	-12.20
		Min	-27.70	-27.72	-12.89	12.89	27.72	27.70	-27.72	-12.89	12.89	27.72	-12.20	12.20	-12.20	12.20
1st Transverse Buckling Mode																
Moment	315	MN-m														
Axial Load	14.1	MN														
	σ_1	Max	1.80	9.78	19.87	19.87	9.78	1.80	9.78	19.87	19.87	9.78	9.09	9.09	9.09	9.09
		Min	-1.80	-9.78	-19.87	-19.87	-9.78	-1.80	-9.78	-19.87	-19.87	-9.78	-9.09	-9.09	-9.09	-9.09
2nd Transverse Buckling Mode																
Moment	114	MN-m														
Axial Load	48.3	MN														
	σ_1	Max	6.16	9.04	12.68	12.68	9.04	6.16	9.04	12.68	12.68	9.04	8.79	8.79	8.79	8.79
		Min	-6.16	-9.04	-12.68	-12.68	-9.04	-6.16	-9.04	-12.68	-12.68	-9.04	-8.79	-8.79	-8.79	-8.79
Envelope of Imperfection Stress																
	σ_1	MIN	-28	-28	-20	-20	-28	-28	-28	-20	-20	-28	-12	-12	-12	-12
		MAX	28	28	20	20	28	28	28	20	20	28	12	12	12	12
Total Stresses:																
IBDAS Forces + Temp + Imperfections																
	σ_1	MIN	-409	-413	-356	-289	-352	-351	-406	-332	-267	-350	-337	-273	-327	-260
		Max	97	96	-7	-125	-89	-91	99	-10	-133	-92	-23	-132	-25	-136
CRITICAL STRESS (capacity)			418	418	397	397	418	418	418	397	397	418	345	345	341	341
CORRESP. REDUCTION FACTOR			1.00	1.00	0.95	0.95	1.00	1.00	1.00	0.95	0.95	1.00	0.82	0.82	0.82	0.82
UTILIZATION RATIO			**0.98**	**0.99**	**0.89**	**0.73**	**0.84**	**0.84**	**0.97**	**0.84**	**0.67**	**0.84**	**0.98**	**0.79**	**0.96**	**0.76**

			LONGITUDINAL STIFFENER STRESS POINTS												
Stress point			LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
		side of cross section	+	+	+	+	+	+	+	+	+	+	+	+	+
Plate ID			TW1	TW1	TW2	TW2	TW2	OW2	OW2	OW2	OW2	OW2	BW1	BW1	BW2
Location of	Y	(mm)	3037.289	3701.208	4365.146	5029.0655	5692.984	6000	6000	6000	6000	6000	3037.289	3701.208	4365.146
Stress point	Z	(mm)	9366.23	8307.12	7247.98	6188.8717	5129.763	2800	1600	1.82E-12	-1600	-2800	-9366.229	-8307.12	-7247.981
IBDAS ULS/SILS SECTION FORCES	σ₁ (Mpa)	min NS	-338	-334	-329	-325	-321	-308	-301	-292	-283	-276	-230	-238	-246
		max NS	-12	-22	-32	-43	-53	-76	-88	-104	-120	-132	-198	-187	-176
		min MY	48	28	8	-12	-32	-76	-99	-129	-160	-182	-306	-286	-267
		max MY	-356	-343	-330	-317	-304	-276	-262	-243	-223	-209	-131	-144	-156
		min MZ	-248	-241	-234	-226	-219	-209	-205	-200	-194	-190	-182	-182	-183
		max MZ	-122	-133	-144	-155	-166	-184	-193	-204	-216	-225	-258	-253	-249
		min VY	-222	-220	-218	-216	-214	-212	-212	-211	-210	-209	-212	-211	-210
		max VY	-134	-139	-145	-151	-157	-168	-173	-181	-188	-194	-220	-216	-212
		min L001	-375	-364	-354	-344	-333	-308	-295	-278	-260	-247	-171	-183	-196
		max L001	53	33	13	-7	-27	-71	-93	-123	-153	-176	-298	-278	-259
		min L002	-373	-364	-356	-348	-339	-318	-306	-291	-276	-264	-193	-205	-217
		max L002	52	34	16	-1	-19	-60	-81	-109	-137	-158	-278	-258	-239
		MIN	-375	-364	-356	-348	-339	-318	-306	-292	-283	-276	-306	-286	-267
		MAX	53	34	16	-1	-19	-60	-81	-104	-120	-132	-131	-144	-156
			min L001	*min L001*	*min L002*	*min L002*	*min L002*	*min L002*	*min L002*	*min NS*	*min NS*	*min NS*	*min MY*	*min MY*	*min MY*
TEMP. LOAD CASE 4500	σ₁ (Mpa)	min NS	4	4	3	2	1	0	-1	-2	-3	-3	-6	-6	-5
		max NS	-4	-3	-2	-2	-1	1	2	3	4	4	7	7	6
		min MY	4	4	3	2	1	0	-1	-2	-3	-3	-6	-6	-5
		max MY	-4	-3	-2	-2	-1	1	2	3	4	4	7	7	6
		min MZ	-4	-3	-2	-2	-1	1	2	3	4	4	7	7	6
		max MZ	4	4	3	2	1	0	-1	-2	-3	-3	-6	-6	-5
		min VY	-4	-3	-2	-2	-1	1	2	3	4	4	7	7	6
		max VY	4	4	3	2	1	0	-1	-2	-3	-3	-6	-6	-5
		min L001	-4	-3	-2	-2	-1	1	2	3	4	4	7	7	6
		max L001	4	4	3	2	1	0	-1	-2	-3	-3	-6	-6	-5
		min L002	-4	-3	-2	-2	-1	1	2	3	4	4	7	7	6
		max L002	4	4	3	2	1	0	-1	-2	-3	-3	-6	-6	-5
		MIN	-4	-3	-2	-2	-1	0	-1	-2	-3	-3	-6	-6	-5
		MAX	4	4	3	2	1	1	2	3	4	4	7	7	6
ULS/SILS + Temp		MIN	-379	-368	-358	-349	-340	-318	-307	-293	-285	-279	-313	-292	-272
		MAX	57	38	19	1	-18	-59	-79	-101	-116	-127	-124	-137	-150

IMPERFECTION STRESSES

1st Longitudinal Buckling

			LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
Moment	1109	MN-m													
Axial Load	0.0	MN													
	σ₁	Max	26.08	23.14	20.19	17.24	14.29	7.80	4.46	0.00	-4.46	-7.80	-26.08	-23.14	-20.19
		Min	-26.08	-23.14	-20.19	-17.24	-14.29	-7.80	-4.46	0.00	4.46	7.80	26.08	23.14	20.19

1st Transverse Buckling Mode

			LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
Moment	315	MN-m													
Axial Load	14.1	MN													
	σ₁	Max	10.98	12.99	14.99	17.00	19.00	19.93	19.93	19.93	19.93	19.93	10.98	12.99	14.99
		Min	-10.98	-12.99	-14.99	-17.00	-19.00	-19.93	-19.93	-19.93	-19.93	-19.93	-10.98	-12.99	-14.99

2nd Transverse Buckling Mode

			LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
Moment	114	MN-m													
Axial Load	48.3	MN													
	σ₁	Max	9.47	10.20	10.92	11.64	12.37	12.70	12.70	12.70	12.70	12.70	9.47	10.20	10.92
		Min	-9.47	-10.20	-10.92	-11.64	-12.37	-12.70	-12.70	-12.70	-12.70	-12.70	-9.47	-10.20	-10.92

Envelope of Imperfection Stress

		LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
σ₁	MIN	-26	-23	-20	-17	-19	-20	-20	-20	-20	-20	-26	-23	-20
	MAX	26	23	20	17	19	20	20	20	20	20	26	23	20

Total Stresses:

IBDAS Forces + Temp + Imperfections

		LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
σ₁	MIN	-405	-391	-379	-367	-359	-338	-327	-313	-305	-299	-339	-316	-292
	Max	83	61	39	18	1	-39	-59	-81	-96	-107	-98	-114	-129

	LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9	LS10	LS11	LS12	LS13	LS14
CRITICAL STRESS (capacity)	409	409	409	409	409	402	378	358	378	402	409	409	409
CORRESP. REDUCTION FACTOR	0.98	0.98	0.98	0.98	0.98	0.96	0.90	0.86	0.90	0.96	0.98	0.98	0.98
UTILIZATION RATIO	**0.99**	**0.95**	**0.93**	**0.90**	**0.88**	**0.84**	**0.86**	**0.88**	**0.81**	**0.74**	**0.83**	**0.77**	**0.71**

			LONGITUDINAL STIFFENER STRESS POINTS												
Stress point			LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23	LS24	LS25	LS26	LS27	LS28
		side of cross section	+	+	+	+	+	+	+	+	+	+	+	+	+
Plate ID			BW2	BW2	TFI1	TFI2	TFI2	TFI2	BFI1	BFI2	BFI2	BFI2	IWT1	IWT1	IWT1
Location of	Y	(mm)	5029.065	5692.984	800	3500	4825	4825	800	3500	4825	4825	2000	2000	2000
Stress point	Z	(mm)	-6188.872	-5129.763	4000	4000	4000	4000	-4000	-4000	-4000	-4000	9000	8000	6667
IBDAS ULS/SILS SECTION FORCES	σ_1 (Mpa)	min NS	-254	-262	-302	-309	-312	-312	-256	-262	-266	-266	-334	-328	-320
		max NS	-166	-155	-66	-65	-65	-65	-145	-144	-144	-144	-16	-26	-39
		min MY	-247	-227	-53	-53	-53	-53	-204	-205	-205	-205	42	23	-3
		max MY	-169	-181	-292	-291	-291	-291	-196	-195	-195	-195	-352	-340	-324
		min MZ	-183	-183	-241	-227	-220	-220	-212	-198	-192	-192	-252	-248	-244
		max MZ	-244	-240	-150	-163	-170	-170	-208	-221	-228	-228	-120	-127	-137
		min VY	-209	-209	-224	-218	-216	-216	-219	-214	-211	-211	-224	-224	-223
		max VY	-208	-204	-155	-159	-161	-161	-192	-196	-197	-197	-134	-139	-145
		min L001	-209	-221	-313	-317	-319	-319	-225	-230	-232	-232	-369	-358	-344
		max L001	-239	-219	-47	-48	-48	-48	-197	-198	-198	-198	46	27	2
		min L002	-229	-241	-315	-322	-326	-326	-238	-246	-250	-250	-366	-357	-344
		max L002	-219	-200	-46	-42	-40	-40	-187	-183	-181	-181	44	26	3
		MIN	-254	-262	-315	-322	-326	-326	-256	-262	-266	-266	-369	-358	-344
		MAX	-166	-155	-46	-42	-40	-40	-145	-144	-144	-144	46	27	3
			min NS	*min NS*	*min L002*	*min L002*	*min L002*	*min L002*	*min NS*	*min NS*	*min NS*	*min NS*	*min L001*	*min L001*	*min L002*
TEMP. LOAD CASE 4500	σ_1 (Mpa)	min NS	-5	-4	2	1	1	1	-3	-3	-4	-4	4	4	3
		max NS	6	6	-2	-1	0	0	3	4	5	5	-4	-4	-3
		min MY	-5	-4	2	1	1	1	-3	-3	-4	-4	4	4	3
		max MY	6	6	-2	-1	0	0	3	4	5	5	-4	-4	-3
		min MZ	6	6	-2	-1	0	0	3	4	5	5	-4	-4	-3
		max MZ	-5	-4	2	1	1	1	-3	-3	-4	-4	4	4	3
		min VY	6	6	-2	-1	0	0	3	4	5	5	-4	-4	-3
		max VY	-5	-4	2	1	1	1	-3	-3	-4	-4	4	4	3
		min L001	6	6	-2	-1	0	0	3	4	5	5	-4	-4	-3
		max L001	-5	-4	2	1	1	1	-3	-3	-4	-4	4	4	3
		min L002	6	6	-2	-1	0	0	3	4	5	5	-4	-4	-3
		max L002	-5	-4	2	1	1	1	-3	-3	-4	-4	4	4	3
		MIN	-5	-4	-2	-1	0	0	-3	-3	-4	-4	-4	-4	-3
		MAX	6	6	2	1	1	1	3	4	5	5	4	4	3
ULS/SILS + Temp		MIN	-259	-266	-317	-323	-326	-326	-259	-266	-269	-269	-373	-362	-347
		MAX	-160	-149	-44	-41	-39	-39	-142	-140	-139	-139	50	31	6

IMPERFECTION STRESSES

1st Longitudinal Buckling

Moment	1109	MN-m													
Axial Load	0.0	MN													
	σ_1	Max	-17.24	-14.29	11.14	11.14	11.14	11.14	-11.14	-11.14	-11.14	-11.14	25.06	22.28	18.57
		Min	17.24	14.29	-11.14	-11.14	-11.14	-11.14	11.14	11.14	11.14	11.14	-25.06	-22.28	-18.57

1st Transverse Buckling Mode

Moment	315	MN-m													
Axial Load	14.1	MN													
	σ_1	Max	17.00	19.00	4.22	12.38	16.38	16.38	4.22	12.38	16.38	16.38	7.85	7.85	7.85
		Min	-17.00	-19.00	-4.22	-12.38	-16.38	-16.38	-4.22	-12.38	-16.38	-16.38	-7.85	-7.85	-7.85

2nd Transverse Buckling Mode

Moment	114	MN-m													
Axial Load	48.3	MN													
	σ_1	Max	11.64	12.37	7.03	9.98	11.42	11.42	7.03	9.98	11.42	11.42	8.34	8.34	8.34
		Min	-11.64	-12.37	-7.03	-9.98	-11.42	-11.42	-7.03	-9.98	-11.42	-11.42	-8.34	-8.34	-8.34

Envelope of Imperfection Stress

σ_1	MIN	-17	-19	-11	-12	-16	-16	-11	-12	-16	-16	-25	-22	-19
	MAX	17	19	11	12	16	16	11	12	16	16	25	22	19

Total Stresses:

IBDAS Forces + Temp + Imperfections

σ_1	MIN	-276	-285	-328	-336	-343	-343	-270	-278	-286	-286	-399	-384	-365
	Max	-142	-130	-33	-28	-23	-23	-131	-128	-123	-123	75	53	24

CRITICAL STRESS (capacity)	409	409	329	342	363	363	329	342	363	363	415	413	412
CORRESP. REDUCTION FACTOR	0.98	0.98	0.79	0.82	0.87	0.87	0.79	0.82	0.87	0.87	0.99	0.99	0.99
UTILIZATION RATIO	**0.67**	**0.70**	**1.00**	**0.98**	**0.94**	**0.94**	**0.82**	**0.81**	**0.79**	**0.79**	**0.96**	**0.93**	**0.89**

			LONGITUDINAL STIFFENER STRESS POINTS												
Stress point			LS29	LS30	LS31	LS32	LS33	LS2	LS3	LS4	LS5	LS6	LS7	LS8	LS9
		side of cross section	+	+	+	+	+	-	-	-	-	-	-	-	-
Plate ID			IWT2	IWB1	IWB2	IWB2	IWB2	TW1	TW1	TW2	TW2	TW2	OW2	OW2	OW2
Location of Stress point	Y	(mm)	2000	2000	2000	2000	2000	-3037.289	-3701.208	-4365.146	-5029.065	-5692.984	-6000	-6000	-6000
	Z	(mm)	5334	-5334	-6667	-8000	-9000	9366.23	8307.12	7247.98	6188.87	5129.76	2800.00	1600.00	0.00
IBDAS ULS/SILS SECTION FORCES	σ1 (Mpa)	min NS	-312	-251	-243	-236	-230	-323	-315	-307	-300	-292	-278	-271	-262
		max NS	-52	-158	-172	-185	-195	-14	-25	-36	-46	-57	-80	-92	-108
		min MY	-28	-230	-255	-280	-299	49	29	9	-11	-31	-75	-97	-128
		max MY	-308	-180	-164	-148	-136	-357	-345	-332	-320	-307	-279	-265	-246
		min MZ	-239	-202	-197	-192	-189	-279	-279	-279	-279	-278	-272	-267	-262
		max MZ	-146	-224	-233	-243	-250	-93	-97	-101	-106	-110	-126	-134	-146
		min VY	-222	-216	-215	-214	-214	-235	-235	-236	-237	-238	-237	-236	-235
		max VY	-151	-200	-206	-212	-217	-125	-129	-133	-137	-141	-151	-157	-164
		min L001	-329	-213	-198	-184	-173	-365	-352	-339	-327	-314	-288	-275	-258
		max L001	-23	-222	-247	-272	-291	54	35	15	-5	-24	-68	-90	-120
		min L002	-331	-229	-216	-203	-194	-356	-344	-332	-320	-308	-284	-273	-258
		max L002	-21	-208	-232	-255	-273	43	24	4	-16	-35	-77	-98	-126
		MIN	-331	-251	-255	-280	-299	-365	-352	-339	-327	-314	-288	-275	-262
		MAX	-21	-158	-164	-148	-136	54	35	15	-5	-24	-68	-90	-108
			min L002	*min NS*	*min MY*	*min MY*	*min MY*	*min L001*	*min L001*	*min L001*	*min L001*	*min L001*	*min L001*	*min L001*	*min MZ*
TEMP. LOAD CASE 4500	σ1 (Mpa)	min NS	2	-4	-5	-5	-6	6	5	5	4	4	2	2	1
		max NS	-2	4	5	6	7	-6	-6	-5	-5	-5	-3	-3	-2
		min MY	2	-4	-5	-5	-6	6	5	5	4	4	2	2	1
		max MY	-2	4	5	6	7	-6	-6	-5	-5	-5	-3	-3	-2
		min MZ	-2	4	5	6	7	-6	-6	-5	-5	-5	-3	-3	-2
		max MZ	2	-4	-5	-5	-6	6	5	5	4	4	2	2	1
		min VY	-2	4	5	6	7	-6	-6	-5	-5	-5	-3	-3	-2
		max VY	2	-4	-5	-5	-6	6	5	5	4	4	2	2	1
		min L001	-2	4	5	6	7	-6	-6	-5	-5	-5	-3	-3	-2
		max L001	2	-4	-5	-5	-6	6	5	5	4	4	2	2	1
		min L002	-2	4	5	6	7	-6	-6	-5	-5	-5	-3	-3	-2
		max L002	2	-4	-5	-5	-6	6	5	5	4	4	2	2	1
		MIN	-2	-4	-5	-5	-6	-6	-6	-5	-5	-5	-3	-3	-2
		MAX	2	4	5	6	7	6	5	5	4	4	2	2	1
ULS/SILS + Temp		MIN	-333	-255	-260	-286	-305	-371	-358	-345	-332	-319	-292	-278	-263
		MAX	-18	-154	-158	-142	-129	60	40	19	-1	-21	-66	-89	-107

IMPERFECTION STRESSES

1st Longitudinal Buckling

Moment	1109	MN-m													
Axial Load	0.0	MN													
	σ1	Max	14.86	-14.86	-18.57	-22.28	-25.06	26.08	23.14	20.19	17.24	14.29	7.80	4.46	0.00
		Min	-14.86	14.86	18.57	22.28	25.06	-26.08	-23.14	-20.19	-17.24	-14.29	-7.80	-4.46	0.00

1st Transverse Buckling Mode

Moment	315	MN-m													
Axial Load	14.1	MN													
	σ1	Max	7.85	7.85	7.85	7.85	7.85	10.98	12.99	14.99	17.00	19.00	19.93	19.93	19.93
		Min	-7.85	-7.85	-7.85	-7.85	-7.85	-10.98	-12.99	-14.99	-17.00	-19.00	-19.93	-19.93	-19.93

2nd Transverse Buckling Mode

Moment	114	MN-m													
Axial Load	48.3	MN													
	σ1	Max	8.34	8.34	8.34	8.34	8.34	9.47	10.20	10.92	11.64	12.37	12.70	12.70	12.70
		Min	-8.34	-8.34	-8.34	-8.34	-8.34	-9.47	-10.20	-10.92	-11.64	-12.37	-12.70	-12.70	-12.70

Envelope of Imperfection Stress

σ1	MIN	-15	-15	-19	-22	-25	-26	-23	-20	-17	-19	-20	-20	-20
	MAX	15	15	19	22	25	26	23	20	17	19	20	20	20

Total Stresses:

IBDAS Forces + Temp + Imperfections

σ1	MIN	-348	-270	-278	-308	-330	-397	-381	-365	-349	-338	-311	-298	-283
	Max	-4	-139	-140	-119	-104	86	63	40	17	-2	-46	-69	-87

CRITICAL STRESS (capacity)	352	352	412	418	418	409	409	409	409	409	402	378	358
CORRESP. REDUCTION FACTOR	0.84	0.84	0.99	1.00	1.00	0.98	0.98	0.98	0.98	0.98	0.96	0.90	0.86
UTILIZATION RATIO	**0.99**	**0.77**	**0.68**	**0.74**	**0.79**	**0.97**	**0.93**	**0.89**	**0.85**	**0.83**	**0.77**	**0.79**	**0.79**

			LONGITUDINAL STIFFENER STRESS POINTS												
Stress point			LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
		side of cross section	-	-	-	-	-	-	-	-	-	-	-	-	-
Plate ID			OW2	OW2	BW1	BW1	BW2	BW2	BW2	TFI1	TFI2	TFI2	TFI2	BFI1	BFI2
Location of	Y	(mm)	-6000	-6000	-3037.289	-3701.208	-4365.146	-5029.065	-5692.984	-800	-3500	-4825	-4825	-800	-3500
Stress point	Z	(mm)	-1600.00	-2800.00	-9366.23	-8307.12	-7247.98	-6188.87	-5129.76	4000.00	4000.00	4000.00	4000.00	-4000.00	-4000.00
IBDAS ULS/SILS SECTION FORCES	σ_1 (Mpa)	min NS	-252	-245	-215	-220	-224	-228	-233	-298	-291	-288	-288	-252	-245
		max NS	-124	-136	-200	-190	-180	-169	-159	-67	-67	-68	-68	-146	-147
		min MY	-158	-181	-306	-285	-265	-245	-225	-53	-52	-52	-52	-204	-204
		max MY	-227	-212	-133	-146	-158	-171	-184	-292	-293	-294	-294	-196	-197
		min MZ	-256	-252	-213	-221	-228	-235	-242	-249	-263	-270	-270	-221	-235
		max MZ	-158	-166	-228	-217	-206	-195	-185	-142	-129	-123	-123	-200	-187
		min VY	-234	-234	-224	-226	-228	-230	-232	-227	-233	-235	-235	-222	-228
		max VY	-172	-177	-211	-206	-200	-194	-188	-153	-149	-148	-148	-190	-186
		min L001	-240	-227	-161	-171	-182	-192	-202	-310	-305	-303	-303	-223	-218
		max L001	-150	-173	-296	-276	-256	-236	-216	-47	-46	-46	-46	-196	-196
		min L002	-242	-231	-176	-184	-193	-201	-209	-310	-303	-299	-299	-234	-226
		max L002	-154	-175	-286	-269	-251	-233	-216	-48	-52	-54	-54	-189	-193
		MIN	-256	-252	-306	-285	-265	-245	-242	-310	-305	-303	-303	-252	-245
		MAX	-124	-136	-133	-146	-158	-169	-159	-47	-46	-46	-46	-146	-147
			min MZ	*min MZ*	*min MY*	*min MY*	*min MY*	*min MY*	*min MZ*	*min L002*	*min L001*	*min L001*	*min L001*	*min NS*	*min NS*
TEMP. LOAD CASE 4500	σ_1 (Mpa)	min NS	0	-1	-5	-4	-4	-3	-2	2	3	3	3	-3	-2
		max NS	-1	0	5	4	3	3	2	-2	-3	-4	-4	3	2
		min MY	0	-1	-5	-4	-4	-3	-2	2	3	3	3	-3	-2
		max MY	-1	0	5	4	3	3	2	-2	-3	-4	-4	3	2
		min MZ	-1	0	5	4	3	3	2	-2	-3	-4	-4	3	2
		max MZ	0	-1	-5	-4	-4	-3	-2	2	3	3	3	-3	-2
		min VY	-1	0	5	4	3	3	2	-2	-3	-4	-4	3	2
		max VY	0	-1	-5	-4	-4	-3	-2	2	3	3	3	-3	-2
		min L001	-1	0	5	4	3	3	2	-2	-3	-4	-4	3	2
		max L001	0	-1	-5	-4	-4	-3	-2	2	3	3	3	-3	-2
		min L002	-1	0	5	4	3	3	2	-2	-3	-4	-4	3	2
		max L002	0	-1	-5	-4	-4	-3	-2	2	3	3	3	-3	-2
		MIN	-1	-1	-5	-4	-4	-3	-2	-2	-3	-4	-4	-3	-2
		MAX	0	0	5	4	3	3	2	2	3	3	3	3	2
ULS/SILS + Temp		MIN	-257	-253	-311	-290	-269	-248	-244	-313	-309	-307	-307	-254	-247
		MAX	-124	-136	-128	-141	-155	-167	-157	-45	-44	-43	-43	-143	-145

IMPERFECTION STRESSES

1st Longitudinal Buckling

			LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
Moment	1109	MN-m													
Axial Load	0.0	MN													
	σ_1	Max	-4.46	-7.80	-26.08	-23.14	-20.19	-17.24	-14.29	11.14	11.14	11.14	11.14	-11.14	-11.14
		Min	4.46	7.80	26.08	23.14	20.19	17.24	14.29	-11.14	-11.14	-11.14	-11.14	11.14	11.14

1st Transverse Buckling Mode

			LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
Moment	315	MN-m													
Axial Load	14.1	MN													
	σ_1	Max	19.93	19.93	10.98	12.99	14.99	17.00	19.00	4.22	12.38	16.38	16.38	4.22	12.38
		Min	-19.93	-19.93	-10.98	-12.99	-14.99	-17.00	-19.00	-4.22	-12.38	-16.38	-16.38	-4.22	-12.38

2nd Transverse Buckling Mode

			LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
Moment	114	MN-m													
Axial Load	48.3	MN													
	σ_1	Max	12.70	12.70	9.47	10.20	10.92	11.64	12.37	7.03	9.98	11.42	11.42	7.03	9.98
		Min	-12.70	-12.70	-9.47	-10.20	-10.92	-11.64	-12.37	-7.03	-9.98	-11.42	-11.42	-7.03	-9.98

Envelope of Imperfection Stress

		LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
σ_1	MIN	-20	-20	-26	-23	-20	-17	-19	-11	-12	-16	-16	-11	-12
	MAX	20	20	26	23	20	17	19	11	12	16	16	11	12

Total Stresses:

IBDAS Forces + Temp + Imperfections

		LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
σ_1	MIN	-277	-273	-337	-313	-289	-265	-263	-324	-321	-323	-323	-265	-259
	Max	-104	-116	-101	-118	-135	-149	-138	-34	-31	-27	-27	-132	-133

	LS10	LS11	LS12	LS13	LS14	LS15	LS16	LS18	LS19	LS20	LS21	LS22	LS23
CRITICAL STRESS (capacity)	378	402	409	409	409	409	409	329	342	363	363	329	342
CORRESP. REDUCTION FACTOR	0.90	0.96	0.98	0.98	0.98	0.98	0.98	0.79	0.82	0.87	0.87	0.79	0.82
UTILIZATION RATIO	**0.73**	**0.68**	**0.82**	**0.76**	**0.71**	**0.65**	**0.64**	**0.99**	**0.94**	**0.89**	**0.89**	**0.81**	**0.76**

			LONGITUDINAL STIFFENER STRESS POINTS									
Stress point			LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
		side of cross section	-	-	-	-	-	-	-	-	-	-
Plate ID			BFI2	BFI2	IWT1	IWT1	IWT1	IWT2	IWB1	IWB2	IWB2	IWB2
Location of	Y	(mm)	-4825	-4825	-2000	-2000	-2000	-2000	-2000	-2000	-2000	-2000
Stress point	Z	(mm)	-4000.00	-4000.00	9000.00	8000.00	6667.00	5334.00	-5334.00	-6667.00	-8000.00	-9000.00
IBDAS ULS/SILS SECTION FORCES	σ_1 (Mpa)	min NS	-242	-242	-323	-318	-310	-302	-241	-233	-226	-220
		max NS	-147	-147	-17	-27	-40	-54	-160	-173	-186	-196
		min MY	-204	-204	42	23	-2	-27	-229	-255	-280	-299
		max MY	-198	-198	-353	-341	-325	-309	-181	-165	-149	-137
		min MZ	-242	-242	-273	-269	-265	-260	-222	-218	-213	-209
		max MZ	-181	-181	-100	-108	-117	-127	-204	-214	-223	-231
		min VY	-231	-231	-232	-232	-231	-230	-224	-223	-223	-222
		max VY	-184	-184	-128	-133	-139	-145	-194	-200	-206	-211
		min L001	-216	-216	-362	-352	-337	-322	-206	-192	-177	-166
		max L001	-195	-195	47	28	3	-22	-221	-246	-271	-290
		min L002	-223	-223	-355	-345	-333	-320	-218	-205	-192	-183
		max L002	-195	-195	38	21	-3	-26	-214	-237	-261	-278
		MIN	-242	-242	-362	-352	-337	-322	-241	-255	-280	-299
		MAX	-147	-147	47	28	3	-22	-160	-165	-149	-137
			min MZ	*min MZ*	*min L001*	*min L001*	*min L001*	*min L001*	*min NS*	*min MY*	*min MY*	*min MY*
TEMP. LOAD CASE 4500	σ_1 (Mpa)	min NS	-2	-2	5	5	4	3	-3	-4	-5	-5
		max NS	1	1	-6	-5	-4	-3	3	4	5	5
		min MY	-2	-2	5	5	4	3	-3	-4	-5	-5
		max MY	1	1	-6	-5	-4	-3	3	4	5	5
		min MZ	1	1	-6	-5	-4	-3	3	4	5	5
		max MZ	-2	-2	5	5	4	3	-3	-4	-5	-5
		min VY	1	1	-6	-5	-4	-3	3	4	5	5
		max VY	-2	-2	5	5	4	3	-3	-4	-5	-5
		min L001	1	1	-6	-5	-4	-3	3	4	5	5
		max L001	-2	-2	5	5	4	3	-3	-4	-5	-5
		min L002	1	1	-6	-5	-4	-3	3	4	5	5
		max L002	-2	-2	5	5	4	3	-3	-4	-5	-5
		MIN	-2	-2	-6	-5	-4	-3	-3	-4	-5	-5
		MAX	1	1	5	5	4	3	3	4	5	5
ULS/SILS + Temp		MIN	-243	-243	-368	-357	-341	-326	-244	-258	-284	-304
		MAX	-146	-146	52	33	7	-19	-157	-161	-144	-131

IMPERFECTION STRESSES

1st Longitudinal Buckling

			LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
Moment	1109	MN-m										
Axial Load	0.0	MN										
	σ_1	Max	-11.14	-11.14	25.06	22.28	18.57	14.86	-14.86	-18.57	-22.28	-25.06
		Min	11.14	11.14	-25.06	-22.28	-18.57	-14.86	14.86	18.57	22.28	25.06

1st Transverse Buckling Mode

			LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
Moment	315	MN-m										
Axial Load	14.1	MN										
	σ_1	Max	16.38	16.38	7.85	7.85	7.85	7.85	7.85	7.85	7.85	7.85
		Min	-16.38	-16.38	-7.85	-7.85	-7.85	-7.85	-7.85	-7.85	-7.85	-7.85

2nd Transverse Buckling Mode

			LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
Moment	114	MN-m										
Axial Load	48.3	MN										
	σ_1	Max	11.42	11.42	8.34	8.34	8.34	8.34	8.34	8.34	8.34	8.34
		Min	-11.42	-11.42	-8.34	-8.34	-8.34	-8.34	-8.34	-8.34	-8.34	-8.34

Envelope of Imperfection Stress

			LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
	σ_1	MIN	-16	-16	-25	-22	-19	-15	-15	-19	-22	-25
		MAX	16	16	25	22	19	15	15	19	22	25

Total Stresses:

IBDAS Forces + Temp + Imperfections

			LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
	σ_1	MIN	-260	-260	-393	-379	-360	-341	-259	-277	-307	-329
		Max	-130	-130	77	55	26	-4	-142	-142	-122	-106

	LS24	LS25	LS26	LS27	LS28	LS29	LS30	LS31	LS32	LS33
CRITICAL STRESS (capacity)	363	363	415	413	412	352	352	412	418	418
CORRESP. REDUCTION FACTOR	0.87	0.87	0.99	0.99	0.99	0.84	0.84	0.99	1.00	1.00
UTILIZATION RATIO	**0.72**	**0.72**	**0.95**	**0.92**	**0.87**	**0.97**	**0.73**	**0.67**	**0.73**	**0.79**

The following tables show the maximum utilization ratios of the previous tables for the unstiffened corners and the longitudinal stiffeners. Therefore, the maximum utilization ratio for Sicilia tower segment 5 under ULS 7 loading is 0.997.

SUMMARY OF CORNER STRESS POINTS

Max UR	0.988
Governing Stress Point	SP2
reduction factor for governing point	1.000
Governing Case for Max Stress Point	min L001
Ns	-2099.36
My	4338.09
Mz	174.17

SUMMARY OF LONGITUDINAL STIFFENER STRESS POINTS

Max UR	0.997
Governing Stress Point	LS18
side of cross section	+
Governing Case for Max Stress Point	min L002
Ns	-2151.48
My	3814.39
Mz	291.96
reduction factor for governing stiff	0.786

5.1.1.6 Overall Verification for Entire Tower for Single Load Combination

The stresses and utilization ratios as shown in the previous section were then calculated for all tower segments. The calculation for each location over the height of the tower was done using the same methods as shown above for segment 5. The following tables show the resulting utilization ratios for all stiffener and corner stress points on the cross-section. Once again this verification of the entire tower leg is shown for the ULS 7 seismic load combination.

For reference, the same values presented in the previous tables for segment 5 can be found in the following tables under analysis section 9, tower segment 5.

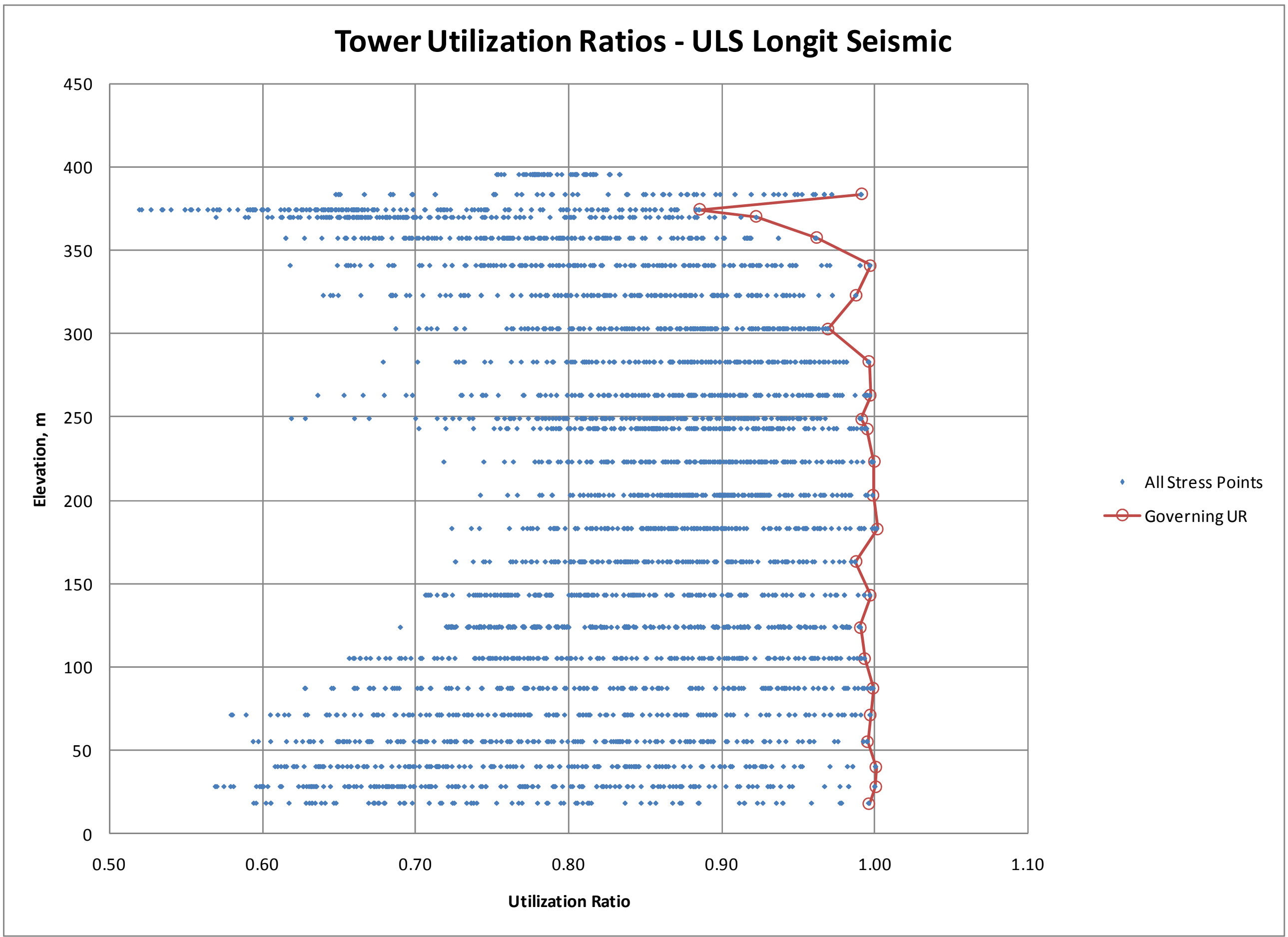
General	Analysis Section			1	2	3	4	5	6	7	8	9	10	11	12	13	14
	S-coordinate			18	28	28	40	40	55	55	71	71	87	87	105	105	123.65
	Cross Section/Segment		Units	1	1	2	2	3	3	4	4	5	5	6	6	7	7
Section Properties	Gross Section	A	(mm^2)	1.09E+07	1.09E+07	1.00E+07	1.00E+07	9.23E+06	9.23E+06	8.58E+06	8.58E+06	7.84E+06	7.84E+06	7.45E+06	7.45E+06	7.53E+06	7.53E+06
		Y	(mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		I_{1_Y}	(mm^4)	5.16E+14	5.16E+14	4.88E+14	4.88E+14	4.61E+14	4.61E+14	4.34E+14	4.34E+14	3.98E+14	3.98E+14	3.62E+14	3.62E+14	3.24E+14	3.24E+14
		I_{2_Z}	(mm^4)	1.57E+14	1.57E+14	1.41E+14	1.41E+14	1.29E+14	1.29E+14	1.18E+14	1.18E+14	1.04E+14	1.04E+14	1.03E+14	1.03E+14	1.14E+14	1.14E+14
VERIFICATION - UTILIZATION RATIOS																	
	Plate Stress Points for Corners	SP1		0.98	0.91	0.98	0.92	0.98	0.91	0.98	0.90	0.98	0.93	1.00	0.96	0.99	0.96
		SP2		1.00	0.93	1.00	0.93	1.00	0.93	1.00	0.90	0.99	0.94	1.00	0.96	0.99	0.98
		SP3		0.79	0.76	0.82	0.78	0.85	0.80	0.86	0.77	0.89	0.86	0.91	0.91	0.87	0.91
		SP4		0.64	0.60	0.66	0.61	0.66	0.62	0.67	0.63	0.73	0.69	0.72	0.71	0.66	0.75
		SP5		0.92	0.87	0.92	0.86	0.92	0.83	0.89	0.77	0.84	0.77	0.84	0.76	0.82	0.84
		SP6		0.93	0.87	0.93	0.86	0.92	0.83	0.89	0.77	0.84	0.77	0.83	0.78	0.83	0.83
		SP7		0.96	0.90	0.97	0.91	0.97	0.89	0.96	0.89	0.97	0.93	0.99	0.96	0.99	0.98
		SP8		0.69	0.66	0.71	0.70	0.75	0.71	0.76	0.73	0.84	0.83	0.88	0.89	0.85	0.88
		SP9		0.67	0.62	0.68	0.61	0.67	0.60	0.66	0.58	0.67	0.69	0.72	0.80	0.74	0.82
		SP10		0.94	0.88	0.94	0.87	0.93	0.83	0.89	0.77	0.84	0.77	0.84	0.80	0.85	0.87
		SP11		0.77	0.75	0.86	0.84	0.93	0.89	0.96	0.91	0.98	0.94	1.00	0.99	0.91	0.91
		SP12		0.61	0.58	0.67	0.64	0.70	0.69	0.75	0.73	0.79	0.77	0.81	0.81	0.74	0.76
		SP13		0.74	0.71	0.82	0.80	0.90	0.87	0.93	0.91	0.96	0.94	1.00	0.99	0.91	0.91
		SP14		0.63	0.60	0.69	0.65	0.73	0.67	0.73	0.72	0.76	0.78	0.82	0.86	0.77	0.82
		Max		**1.00**	**0.93**	**1.00**	**0.93**	**1.001**	**0.93**	**1.00**	**0.91**	**0.99**	**0.94**	**1.00**	**0.99**	**0.99**	**0.98**
	Governing Stress Point			**SP2**	**SP2**	**SP2**	**SP2**	**SP2**	**SP2**	**SP2**	**SP13**	**SP2**	**SP13**	**SP2**	**SP13**	**SP7**	**SP7**
	reduction factor			**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**0.79**	**1.00**	**0.82**	**1.00**	**0.82**	**1.00**	**1.00**
	Govern. Case for Max Stress Point			**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**
		Ns		**-1976**	**-1974**	**-1974**	**-2038**	**-2038**	**-2065**	**-2067**	**-2108**	**-2099**	**-2094**	**-2092**	**-2038**	**-2036**	**-2002**
		My		**10243**	**8913**	**8910**	**7287**	**7286**	**5711**	**5720**	**4284**	**4338**	**3812**	**3835**	**3886**	**3898**	**3772**
		Mz		**431**	**485**	**487**	**437**	**437**	**408**	**407**	**183**	**174**	**92**	**76**	**-36**	**-42**	**-39**
	Longitudinal Stiffener Stress Points side +	LS2		0.98	0.92	0.98	0.92	0.99	0.92	0.99	0.90	0.99	0.94	0.99	0.95	0.98	0.98
		LS3		0.92	0.87	0.93	0.88	0.94	0.88	0.95	0.87	0.95	0.91	0.95	0.93	0.95	0.96
		LS4		0.87	0.82	0.88	0.83	0.89	0.84	0.92	0.84	0.93	0.88	0.93	0.92	0.93	0.95
		LS5		0.84	0.79	0.85	0.81	0.88	0.82	0.89	0.81	0.90	0.86	0.90	0.90	0.91	0.94
		LS6		0.81	0.77	0.83	0.79	0.86	0.81	0.88	0.79	0.88	0.84	0.88	0.89	0.89	0.93
		LS7		0.74	0.71	0.78	0.74	0.80	0.76	0.82	0.75	0.84	0.81	0.83	0.84	0.83	0.88
		LS8		0.69	0.67	0.74	0.70	0.78	0.75	0.83	0.76	0.86	0.84	0.85	0.86	0.80	0.86
		LS9		0.63	0.63	0.70	0.67	0.76	0.73	0.82	0.76	0.88	0.85	0.86	0.87	0.76	0.82
		LS10		0.61	0.60	0.65	0.64	0.71	0.69	0.76	0.71	0.81	0.77	0.78	0.79	0.72	0.78
		LS11		0.62	0.58	0.63	0.62	0.68	0.66	0.71	0.66	0.74	0.70	0.72	0.71	0.69	0.75
		LS12		0.88	0.84	0.89	0.83	0.89	0.80	0.87	0.76	0.83	0.76	0.81	0.74	0.79	0.82
		LS13		0.81	0.77	0.82	0.77	0.82	0.74	0.80	0.70	0.77	0.71	0.76	0.70	0.74	0.78
		LS14		0.73	0.70	0.74	0.70	0.74	0.68	0.74	0.65	0.71	0.66	0.70	0.66	0.69	0.73
		LS15		0.67	0.64	0.68	0.64	0.69	0.63	0.68	0.61	0.67	0.65	0.67	0.67	0.66	0.74
		LS16		0.64	0.60	0.66	0.61	0.66	0.62	0.67	0.63	0.70	0.66	0.69	0.68	0.67	0.76
		LS18		0.75	0.73	0.86	0.84	0.95	0.91	1.00	0.96	1.00	0.96	0.98	0.96	0.91	0.90
		LS19		0.80	0.77	0.88	0.85	0.92	0.88	0.96	0.89	0.98	0.95	0.97	0.96	0.89	0.91
		LS20		0.81	0.77	0.87	0.84	0.91	0.86	0.94	0.86	0.94	0.91	0.93	0.93	0.88	0.92
		LS21		0.81	0.77	0.87	0.84	0.91	0.86	0.94	0.86	0.94	0.91	0.93	0.93	0.88	0.92
		LS22		0.59	0.57	0.67	0.64	0.72	0.72	0.79	0.79	0.82	0.81	0.82	0.82	0.76	0.78
		LS23		0.60	0.57	0.66	0.66	0.71	0.70	0.77	0.74	0.81	0.79	0.80	0.80	0.73	0.75
		LS24		0.63	0.60	0.68	0.65	0.71	0.69	0.76	0.72	0.79	0.75	0.76	0.76	0.70	0.74
		LS25		0.63	0.60	0.68	0.65	0.71	0.69	0.76	0.72	0.79	0.75	0.76	0.76	0.70	0.74
		LS26		0.94	0.88	0.95	0.89	0.95	0.89	0.96	0.88	0.96	0.91	0.97	0.93	0.97	0.96
		LS27		0.89	0.83	0.90	0.84	0.91	0.85	0.92	0.85	0.93	0.89	0.94	0.91	0.94	0.93
		LS28		0.81	0.77	0.83	0.79	0.85	0.80	0.87	0.81	0.89	0.85	0.90	0.87	0.90	0.90
		LS29		0.77	0.73	0.79	0.76	0.86	0.82	0.94	0.89	0.99	0.95	0.99	0.97	0.97	0.97
		LS30		0.63	0.60	0.64	0.61	0.68	0.63	0.72	0.69	0.77	0.74	0.77	0.78	0.75	0.78
		LS31		0.70	0.66	0.71	0.67	0.71	0.65	0.70	0.62	0.68	0.63	0.67	0.66	0.66	0.69
		LS32		0.79	0.75	0.80	0.74	0.79	0.72	0.77	0.68	0.74	0.68	0.73	0.68	0.72	0.74
		LS33		0.85	0.81	0.86	0.80	0.86	0.78	0.83	0.72	0.79	0.73	0.79	0.72	0.77	0.79
	Longitudinal Stiffener Stress Points side -	LS2		0.94	0.88	0.94	0.89	0.95	0.88	0.95	0.88	0.97	0.93	0.98	0.96	0.98	0.98
		LS3		0.87	0.83	0.89	0.84	0.90	0.83	0.90	0.85	0.93	0.90	0.94	0.94	0.95	0.96
		LS4		0.81	0.77	0.82	0.79	0.84	0.79	0.85	0.81	0.89	0.86	0.91	0.92	0.93	0.94
		LS5		0.76	0.73	0.78	0.75	0.81	0.75	0.82	0.77	0.85	0.84	0.89	0.89	0.90	0.92
		LS6		0.72	0.68	0.74	0.71	0.77	0.72	0.79	0.75	0.83	0.82	0.86	0.87	0.88	0.90
		LS7		0.67	0.63	0.69	0.64	0.70	0.66	0.71	0.70	0.77	0.78	0.80	0.84	0.82	0.88
		LS8		0.68	0.63	0.69	0.64	0.71	0.65	0.72	0.70	0.79	0.81	0.83	0.87	0.81	0.87
		LS9		0.68	0.63	0.69	0.63	0.72	0.65	0.74	0.69	0.79	0.83	0.85	0.90	0.79	0.87
		LS10		0.68	0.63	0.69	0.63	0.70	0.63	0.70	0.64	0.73	0.76	0.78	0.84	0.77	0.85
		LS11		0.67	0.63	0.69	0.62	0.68	0.61	0.67	0.60	0.68	0.70	0.72	0.78	0.76	0.84
		LS12		0.91	0.85	0.91	0.84	0.90	0.81	0.87	0.75	0.82	0.76	0.81	0.79	0.83	0.86
		LS13		0.85	0.79	0.84	0.78	0.83	0.75	0.81	0.70	0.76	0.71	0.76	0.75	0.78	0.82
		LS14		0.78	0.73	0.78	0.71	0.76	0.69	0.74	0.64	0.71	0.66	0.70	0.75	0.74	0.81
		LS15		0.72	0.68	0.73	0.67	0.71	0.64	0.69	0.59	0.65	0.65	0.67	0.76	0.75	0.82
		LS16		0.69	0.63	0.68	0.62	0.67	0.59	0.66	0.58	0.64	0.66	0.68	0.77	0.75	0.83
		LS18		0.74	0.71	0.84	0.82	0.93	0.89	0.97	0.94	0.99	0.96	0.98	0.96	0.91	0.90
		LS19		0.73	0.70	0.80	0.78	0.84	0.81	0.88	0.86	0.94	0.93	0.95	0.95	0.89	0.90
		LS20		0.71	0.68	0.77	0.75	0.81	0.77	0.84	0.81	0.89	0.88	0.90	0.91	0.87	0.89
		LS21		0.71	0.68	0.77	0.75	0.81	0.77	0.84	0.81	0.89	0.88	0.90	0.91	0.87	0.89
		LS22		0.60	0.57	0.68	0.64	0.72	0.70	0.77	0.77	0.81	0.81	0.82	0.83	0.77	0.80
		LS23		0.65	0.61	0.71	0.65	0.71	0.65	0.73	0.69	0.76	0.79	0.80	0.84	0.77	0.83
		LS24		0.68	0.64	0.72	0.66	0.72	0.65	0.73	0.65	0.72	0.75	0.76	0.82	0.76	0.84
		LS25		0.68	0.64	0.72	0.66	0.72	0.65	0.73	0.65	0.72	0.75	0.76	0.82	0.76	0.84
		LS26		0.91	0.86	0.92	0.86	0.93	0.86	0.93	0.86	0.95	0.91	0.97	0.94	0.98	0.96
		LS27		0.86	0.81	0.87	0.82	0.88	0.82	0.89	0.84	0.92	0.88	0.94	0.91	0.94	0.93
		LS28		0.79	0.75	0.80	0.76	0.82	0.77	0.84	0.80	0.87	0.84	0.90	0.88	0.90	0.90
		LS29		0.74	0.70	0.76	0.73	0.83	0.79	0.90	0.87	0.97	0.94	0.99	0.97	0.97	0.97
		LS30		0.65	0.61	0.66	0.62	0.69	0.63	0.72	0.66	0.73	0.74	0.77	0.80	0.77	0.82
		LS31		0.72	0.67	0.72	0.67	0.72	0.65	0.71	0.61	0.67	0.63	0.67	0.68	0.69	0.72
		LS32		0.80	0.76	0.81	0.75	0.80	0.73	0.78	0.67	0.73	0.68	0.73	0.70	0.74	0.76
		LS33		0.87	0.82	0.87	0.81	0.87	0.78	0.84	0.72	0.79	0.73	0.79	0.75	0.80	0.81
		MAX		**0.98**	**0.92**	**0.98**	**0.92**	**0.99**	**0.92**	**1.00**	**0.96**	**0.997**	**0.963**	**0.993**	**0.975**	**0.983**	**0.983**
	Governing Stiffener			**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS18**	**LS18**	**LS18**	**LS18**	**LS29**	**LS29**	**LS2**	**LS2**
		side		**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**-**	**-**	**-**
	reduction factor for governing			**0.99**	**0.99**	**0.99**	**0.99**	**0.99**	**0.99**	**0.75**	**0.75**	**0.786**	**0.786**	**0.853**	**0.853**	**0.989**	**0.989**
	Governing Case for Max Stiffener			**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L002**	**min L002**	**min L002**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**
		Ns		**-1976**	**-1974**	**-1974**	**-2038**	**-2038**	**-2065**	**-2124**	**-2152**	**-2151**	**-2094**	**-2092**	**-2038**	**-2036**	**-2002**
		My		**10243**	**8913**	**8910**	**7287**	**7286**	**5711**	**5073**	**3811**	**3814**	**3812**	**3835**	**3886**	**3898**	**3772**
		Mz		**431**	**485**	**487**	**437**	**437**	**408**	**585**	**292**	**292**	**92**	**76**	**-36**	**-42**	**-39**
Summary	Plates	UR		**1.00**	**0.93**	**1.00**	**0.93**	**1.00**	**0.93**	**1.00**	**0.91**	**0.99**	**0.94**	**1.00**	**0.99**	**0.99**	**0.98**
	Stiffeners	UR		**0.98**	**0.92**	**0.98**	**0.92**	**0.99**	**0.92**	**1.00**	**0.96**	**1.00**	**0.96**	**0.99**	**0.97**	**0.98**	**0.98**
	Overall Max	**UR**		**1.00**	**0.93**	**1.00**	**0.93**	**1.00**	**0.93**	**1.00**	**0.96**	**1.00**	**0.96**	**1.00**	**0.99**	**0.99**	**0.98**

General	Analysis Section			15	16	17	18	19	20	21	22	23	24	25	26	27	28
	S-coordinate			123.65	124	124	143	143	163	163	183	183	203	203	223	223	243
	Cross Section/Segment		Units	8	8	8	8	9	9	10	10	11	11	12	12	13	13
Section Properties	Gross Section	A	(mm^2)	7.88E+06	7.88E+06	7.88E+06	7.88E+06	7.45E+06	7.45E+06	7.091E+06	7.09E+06	6.98E+06	6.98E+06	7.19E+06	7.19E+06	7.52E+06	7.52E+06
		Y	(mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		I_{1_Y}	(mm^4)	3.11E+14	3.11E+14	3.11E+14	3.11E+14	3.02E+14	3.02E+14	3.08E+14	3.08E+14	3.10E+14	3.10E+14	3.11E+14	3.11E+14	3.27E+14	3.27E+14
		I_{2_Z}	(mm^4)	1.31E+14	1.31E+14	1.31E+14	1.31E+14	1.20E+14	1.20E+14	1.07E+14	1.07E+14	1.02E+14	1.02E+14	1.08E+14	1.08E+14	1.15E+14	1.15E+14
VERIFICATION - UTILIZATION RATIOS																	
	Plate Stress Points for Corners	SP1		0.98	0.98	0.98	0.95	0.99	0.98	0.96	0.98	0.98	0.98	0.97	0.99	0.94	0.99
		SP2		0.98	0.98	0.98	0.94	0.99	0.95	0.98	0.99	1.00	1.00	0.98	1.00	0.95	0.99
		SP3		0.92	0.92	0.92	0.88	0.93	0.86	0.91	0.84	0.85	0.87	0.84	0.86	0.82	0.86
		SP4		0.74	0.74	0.74	0.74	0.79	0.78	0.82	0.79	0.80	0.85	0.82	0.85	0.81	0.86
		SP5		0.85	0.85	0.85	0.86	0.90	0.90	0.90	0.91	0.91	0.93	0.91	0.96	0.92	0.96
		SP6		0.88	0.88	0.88	0.89	0.93	0.94	0.90	0.90	0.91	0.93	0.91	0.97	0.93	0.98
		SP7		0.94	0.94	0.94	0.90	0.94	0.92	0.94	0.96	0.97	0.97	0.95	0.98	0.93	0.98
		SP8		0.82	0.82	0.82	0.77	0.81	0.77	0.80	0.76	0.77	0.80	0.78	0.80	0.76	0.79
		SP9		0.78	0.78	0.78	0.71	0.76	0.75	0.78	0.78	0.79	0.85	0.83	0.89	0.85	0.89
		SP10		0.85	0.85	0.85	0.87	0.90	0.88	0.89	0.90	0.90	0.93	0.91	0.98	0.93	0.99
		SP11		0.90	0.90	0.90	0.87	0.92	0.87	0.91	0.88	0.89	0.88	0.86	0.85	0.81	0.82
		SP12		0.75	0.75	0.75	0.76	0.80	0.81	0.86	0.86	0.87	0.90	0.87	0.90	0.86	0.87
		SP13		0.85	0.85	0.85	0.83	0.87	0.84	0.87	0.85	0.86	0.86	0.84	0.84	0.80	0.82
		SP14		0.72	0.72	0.72	0.74	0.79	0.80	0.84	0.86	0.87	0.90	0.87	0.91	0.87	0.90
		Max		**0.98**	**0.98**	**0.98**	**0.95**	**0.99**	**0.98**	**0.98**	**0.99**	**1.00**	**1.00**	**0.98**	**1.00**	**0.95**	**0.99**
	Governing Stress Point			**SP2**	**SP2**	**SP2**	**SP1**	**SP1**	**SP1**	**SP2**	**SP2**	**SP2**	**SP2**	**SP2**	**SP2**	**SP2**	**SP2**
	reduction factor			**1.00**	**1.00**	**1.00**	**0.95**	**0.95**	**0.95**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
	Govern. Case for Max Stress Point			**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**
		Ns		**-1986**	**-1986**	**-1985**	**-1931**	**-1947**	**-1855**	**-1857**	**-1830**	**-1831**	**-1765**	**-1765**	**-1681**	**-1681**	**-1647**
		My		**3697**	**3692**	**3694**	**3591**	**3481**	**3576**	**3571**	**3588**	**3586**	**3625**	**3626**	**4122**	**4124**	**4806**
		Mz		**654**	**652**	**652**	**362**	**472**	**324**	**322**	**270**	**271**	**221**	**221**	**168**	**166**	**103**
	Longitudinal Stiffener Stress Points side +	LS2		0.99	0.99	0.99	0.95	1.00	0.95	0.99	0.99	1.00	1.00	0.98	1.00	0.95	0.99
		LS3		0.97	0.97	0.97	0.93	0.98	0.93	0.97	0.96	0.96	0.96	0.94	0.95	0.90	0.94
		LS4		0.96	0.95	0.95	0.91	0.98	0.92	0.96	0.93	0.95	0.94	0.92	0.93	0.88	0.91
		LS5		0.94	0.94	0.94	0.90	0.97	0.90	0.95	0.90	0.91	0.92	0.90	0.91	0.86	0.88
		LS6		0.94	0.94	0.94	0.90	0.96	0.89	0.94	0.87	0.89	0.91	0.88	0.89	0.85	0.88
		LS7		0.90	0.89	0.89	0.85	0.91	0.84	0.89	0.80	0.82	0.84	0.81	0.85	0.81	0.85
		LS8		0.87	0.87	0.87	0.82	0.88	0.81	0.88	0.79	0.82	0.85	0.79	0.84	0.79	0.84
		LS9		0.84	0.84	0.84	0.79	0.84	0.78	0.86	0.79	0.84	0.87	0.78	0.83	0.78	0.83
		LS10		0.80	0.80	0.80	0.76	0.81	0.78	0.84	0.79	0.83	0.86	0.80	0.84	0.79	0.85
		LS11		0.77	0.77	0.77	0.75	0.80	0.78	0.83	0.79	0.80	0.84	0.81	0.85	0.80	0.86
		LS12		0.84	0.84	0.84	0.85	0.89	0.88	0.90	0.89	0.90	0.93	0.91	0.97	0.92	0.96
		LS13		0.79	0.79	0.79	0.81	0.84	0.83	0.85	0.86	0.87	0.90	0.88	0.93	0.88	0.91
		LS14		0.74	0.74	0.74	0.76	0.80	0.79	0.83	0.85	0.87	0.90	0.87	0.92	0.87	0.89
		LS15		0.72	0.72	0.72	0.73	0.79	0.79	0.84	0.83	0.85	0.89	0.86	0.90	0.86	0.87
		LS16		0.74	0.74	0.74	0.75	0.80	0.79	0.84	0.82	0.84	0.88	0.86	0.89	0.84	0.88
		LS18		0.86	0.86	0.86	0.83	0.90	0.87	0.96	0.94	0.96	0.94	0.92	0.90	0.82	0.83
		LS19		0.88	0.88	0.88	0.86	0.93	0.87	0.95	0.90	0.91	0.91	0.87	0.87	0.81	0.83
		LS20		0.92	0.92	0.92	0.88	0.95	0.89	0.95	0.88	0.89	0.91	0.87	0.88	0.83	0.86
		LS21		0.92	0.92	0.92	0.88	0.95	0.89	0.95	0.88	0.89	0.91	0.87	0.88	0.83	0.86
		LS22		0.73	0.73	0.73	0.75	0.81	0.83	0.91	0.94	0.95	0.97	0.94	0.97	0.88	0.89
		LS23		0.74	0.74	0.74	0.75	0.81	0.81	0.89	0.88	0.89	0.92	0.87	0.90	0.85	0.86
		LS24		0.76	0.76	0.76	0.76	0.82	0.81	0.87	0.85	0.87	0.90	0.86	0.89	0.84	0.86
		LS25		0.76	0.76	0.76	0.76	0.82	0.81	0.87	0.85	0.87	0.90	0.86	0.89	0.84	0.86
		LS26		0.95	0.95	0.95	0.91	0.97	0.94	0.98	0.99	1.00	0.99	0.98	0.99	0.95	0.99
		LS27		0.94	0.94	0.94	0.91	0.97	0.93	0.97	0.98	0.99	0.98	0.95	0.96	0.94	0.98
		LS28		0.92	0.92	0.92	0.89	0.95	0.91	0.96	0.95	0.96	0.95	0.90	0.91	0.91	0.93
		LS29		0.96	0.95	0.95	0.92	0.98	0.93	0.94	0.93	0.94	0.92	0.90	0.90	0.88	0.90
		LS30		0.76	0.76	0.76	0.78	0.84	0.85	0.86	0.89	0.90	0.92	0.90	0.93	0.91	0.93
		LS31		0.72	0.72	0.72	0.74	0.79	0.80	0.85	0.89	0.90	0.93	0.88	0.91	0.92	0.94
		LS32		0.76	0.76	0.76	0.78	0.81	0.81	0.82	0.86	0.87	0.90	0.86	0.91	0.89	0.92
		LS33		0.79	0.79	0.79	0.81	0.84	0.84	0.85	0.86	0.87	0.90	0.88	0.93	0.89	0.93
	Longitudinal Stiffener Stress Points side -	LS2		0.94	0.94	0.94	0.90	0.94	0.91	0.94	0.95	0.96	0.96	0.95	0.98	0.93	0.97
		LS3		0.92	0.91	0.91	0.88	0.91	0.88	0.91	0.91	0.92	0.92	0.91	0.92	0.88	0.92
		LS4		0.89	0.89	0.89	0.84	0.89	0.86	0.90	0.88	0.89	0.90	0.88	0.90	0.85	0.89
		LS5		0.86	0.86	0.86	0.82	0.87	0.83	0.87	0.83	0.85	0.87	0.85	0.87	0.83	0.86
		LS6		0.84	0.84	0.84	0.79	0.84	0.80	0.84	0.80	0.81	0.85	0.82	0.84	0.80	0.83
		LS7		0.78	0.78	0.78	0.72	0.76	0.73	0.76	0.72	0.74	0.77	0.74	0.76	0.72	0.74
		LS8		0.78	0.78	0.78	0.71	0.75	0.73	0.78	0.74	0.77	0.82	0.76	0.79	0.74	0.76
		LS9		0.79	0.79	0.79	0.71	0.75	0.74	0.81	0.78	0.82	0.88	0.79	0.83	0.78	0.80
		LS10		0.79	0.79	0.79	0.71	0.76	0.74	0.80	0.78	0.81	0.87	0.81	0.85	0.81	0.83
		LS11		0.79	0.79	0.79	0.71	0.76	0.75	0.79	0.78	0.79	0.85	0.82	0.87	0.83	0.86
		LS12		0.84	0.84	0.84	0.86	0.89	0.87	0.88	0.88	0.88	0.93	0.91	0.98	0.94	0.99
		LS13		0.80	0.80	0.80	0.81	0.85	0.82	0.83	0.85	0.86	0.90	0.89	0.95	0.91	0.96
		LS14		0.75	0.75	0.75	0.76	0.81	0.77	0.81	0.84	0.86	0.90	0.88	0.95	0.90	0.94
		LS15		0.75	0.75	0.75	0.72	0.77	0.77	0.81	0.82	0.84	0.90	0.87	0.94	0.89	0.93
		LS16		0.78	0.78	0.78	0.72	0.77	0.76	0.81	0.81	0.83	0.89	0.86	0.93	0.88	0.92
		LS18		0.84	0.84	0.84	0.82	0.89	0.85	0.94	0.93	0.94	0.93	0.91	0.90	0.81	0.83
		LS19		0.81	0.81	0.81	0.79	0.84	0.81	0.88	0.85	0.86	0.87	0.83	0.84	0.79	0.81
		LS20		0.82	0.82	0.82	0.78	0.84	0.80	0.85	0.82	0.83	0.85	0.81	0.82	0.78	0.80
		LS21		0.82	0.82	0.82	0.78	0.84	0.80	0.85	0.82	0.83	0.85	0.81	0.82	0.78	0.80
		LS22		0.72	0.72	0.72	0.74	0.80	0.82	0.91	0.94	0.95	0.97	0.94	0.97	0.88	0.90
		LS23		0.73	0.73	0.73	0.72	0.78	0.79	0.86	0.87	0.88	0.92	0.88	0.92	0.87	0.90
		LS24		0.77	0.77	0.77	0.72	0.78	0.79	0.84	0.84	0.85	0.91	0.87	0.92	0.87	0.90
		LS25		0.77	0.77	0.77	0.72	0.78	0.79	0.84	0.84	0.85	0.91	0.87	0.92	0.87	0.90
		LS26		0.91	0.91	0.91	0.88	0.94	0.92	0.95	0.96	0.97	0.97	0.96	0.98	0.94	0.98
		LS27		0.90	0.90	0.90	0.88	0.93	0.91	0.94	0.95	0.96	0.96	0.93	0.94	0.93	0.97
		LS28		0.88	0.88	0.88	0.86	0.91	0.88	0.92	0.93	0.94	0.93	0.88	0.89	0.89	0.92
		LS29		0.90	0.90	0.90	0.88	0.94	0.90	0.91	0.90	0.91	0.90	0.88	0.88	0.86	0.89
		LS30		0.73	0.74	0.74	0.76	0.82	0.84	0.85	0.88	0.89	0.92	0.90	0.94	0.92	0.95
		LS31		0.72	0.72	0.72	0.74	0.79	0.79	0.84	0.89	0.90	0.93	0.88	0.93	0.93	0.97
		LS32		0.76	0.76	0.76	0.78	0.82	0.80	0.81	0.86	0.87	0.91	0.87	0.92	0.90	0.95
		LS33		0.80	0.80	0.80	0.81	0.85	0.83	0.84	0.85	0.86	0.90	0.88	0.95	0.90	0.95
		MAX		**0.991**	**0.990**	**0.990**	**0.946**	**0.997**	**0.953**	**0.987**	**0.991**	**1.000**	**0.995**	**0.981**	**0.998**	**0.955**	**0.994**
	Governing Stiffener			**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS2**	**LS26**	**LS26**
	side			**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**	**+**
	reduction factor for governing			**0.983**	**0.983**	**0.983**	**0.983**	**0.984**	**0.984**	**0.982**	**0.982**	**0.982**	**0.982**	**0.976**	**0.976**	**0.954**	**0.954**
	Governing Case for Max Stiffener			**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**	**min L001**
		Ns		**-1986**	**-1986**	**-1985**	**-1931**	**-1947**	**-1855**	**-1857**	**-1830**	**-1831**	**-1765**	**-1765**	**-1681**	**-1681**	**-1647**
		My		**3697**	**3692**	**3694**	**3591**	**3481**	**3576**	**3571**	**3588**	**3586**	**3625**	**3626**	**4122**	**4124**	**4806**
		Mz		**654**	**652**	**652**	**362**	**472**	**324**	**322**	**270**	**271**	**221**	**221**	**168**	**166**	**103**
Summary	**Plates**	**UR**		**0.98**	**0.98**	**0.98**	**0.95**	**0.99**	**0.98**	**0.98**	**0.99**	**1.00**	**1.00**	**0.98**	**1.00**	**0.95**	**0.99**
	Stiffeners	**UR**		**0.99**	**0.99**	**0.99**	**0.95**	**1.00**	**0.95**	**0.99**	**0.99**	**1.00**	**1.00**	**0.98**	**1.00**	**0.95**	**0.99**
	Overall Max	**UR**		**0.99**	**0.99**	**0.99**	**0.95**	**1.00**	**0.98**	**0.99**	**0.99**	**1.00**	**1.00**	**0.98**	**1.00**	**0.95**	**0.99**

General	Analysis Section			29	30	31	32	33	34	35	36	37	38	39	40	41	42
	S-coordinate			243	249.0	249.0	263	263	283	283	303	303	323	323	341	341	357.4
	Cross Section/Segment		Units	14	14	14	14	15	15	16	16	17	17	18	18	19	19
Section Properties	Gross Section	A	(mm^2)	7.88E+06	7.88E+06	7.88E+06	7.88E+06	7.72E+06	7.72E+06	7.50E+06	7.50E+06	7.39E+06	7.39E+06	7.22E+06	7.22E+06	7.14E+06	7.14E+06
		Y	(mm)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
		I_{1_Y}	(mm^4)	3.51E+14	3.51E+14	3.51E+14	3.51E+14	3.66E+14	3.66E+14	3.71E+14	3.71E+14	3.69E+14	3.69E+14	3.42E+14	3.42E+14	3.00E+14	3.00E+14
		I_{2_Z}	(mm^4)	1.19E+14	1.19E+14	1.19E+14	1.19E+14	1.11E+14	1.11E+14	1.02E+14	1.02E+14	1.00E+14	1.00E+14	1.01E+14	1.01E+14	1.11E+14	1.11E+14
VERIFICATION - UTILIZATION RATIOS																	
	Plate Stress Points for Corners	SP1		0.93	0.94	0.94	0.96	0.95	0.95	0.96	0.93	0.94	0.89	0.93	0.85	0.92	0.82
		SP2		0.94	0.95	0.97	0.98	0.97	0.96	0.97	0.94	0.95	0.88	0.92	0.83	0.89	0.78
		SP3		0.82	0.82	0.84	0.84	0.86	0.80	0.87	0.77	0.78	0.70	0.68	0.67	0.68	0.70
		SP4		0.82	0.77	0.90	0.90	0.91	0.85	0.93	0.82	0.83	0.73	0.72	0.68	0.71	0.67
		SP5		0.91	0.92	0.99	1.00	0.99	0.98	1.00	0.95	0.96	0.90	0.94	0.86	0.91	0.81
		SP6		0.92	0.93	0.95	0.97	0.96	0.97	0.98	0.96	0.97	0.92	0.96	0.90	0.97	0.88
		SP7		0.93	0.94	0.92	0.94	0.93	0.93	0.94	0.93	0.94	0.90	0.94	0.87	0.92	0.82
		SP8		0.75	0.75	0.71	0.74	0.74	0.73	0.79	0.76	0.77	0.74	0.73	0.74	0.76	0.80
		SP9		0.84	0.85	0.74	0.77	0.78	0.79	0.85	0.83	0.85	0.85	0.83	0.88	0.88	0.92
		SP10		0.94	0.95	0.92	0.94	0.93	0.95	0.96	0.96	0.97	0.95	0.99	0.94	0.99	0.92
		SP11		0.78	0.78	0.81	0.80	0.88	0.86	0.81	0.77	0.84	0.79	0.82	0.78	0.74	0.71
		SP12		0.82	0.82	0.88	0.88	0.97	0.94	0.89	0.84	0.92	0.87	0.90	0.85	0.81	0.78
		SP13		0.78	0.78	0.75	0.77	0.84	0.83	0.80	0.78	0.84	0.82	0.85	0.82	0.78	0.76
		SP14		0.85	0.86	0.81	0.83	0.90	0.90	0.87	0.86	0.93	0.92	0.95	0.95	0.89	0.89
		Max		**0.94**	**0.95**	**0.99**	**1.00**	**0.99**	**0.98**	**1.00**	**0.96**	**0.97**	**0.95**	**0.99**	**0.95**	**0.99**	**0.92**
	Governing Stress Point			**SP2**	**SP10**	**SP5**	**SP5**	**SP5**	**SP5**	**SP5**	**SP10**	**SP10**	**SP10**	**SP10**	**SP14**	**SP10**	**SP9**
	reduction factor			**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**0.82**	**1.00**	**1.00**
	Govern. Case for Max Stress Point			**min L001**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**
		Ns		**-1642**	**-1952**	**-1918**	**-1902**	**-1902**	**-1879**	**-1879**	**-1856**	**-1856**	**-1833**	**-1833**	**-1813**	**-1812**	**-1795**
		My		**4806**	**-3388**	**-3865**	**-4158**	**-4158**	**-4389**	**-4389**	**-4293**	**-4294**	**-3954**	**-3954**	**-3399**	**-3400**	**-2599**
		Mz		**105**	**-331**	**672**	**520**	**520**	**261**	**261**	**-46**	**-46**	**-379**	**-379**	**-680**	**-680**	**-954**
	Longitudinal Stiffener Stress Points side +	LS2		0.93	0.94	0.96	0.97	0.96	0.96	0.97	0.93	0.94	0.87	0.90	0.81	0.88	0.79
		LS3		0.88	0.89	0.92	0.92	0.92	0.91	0.92	0.88	0.89	0.82	0.85	0.77	0.83	0.76
		LS4		0.87	0.88	0.91	0.92	0.92	0.89	0.93	0.87	0.88	0.81	0.83	0.76	0.81	0.76
		LS5		0.84	0.85	0.89	0.89	0.90	0.87	0.91	0.83	0.84	0.76	0.78	0.73	0.77	0.73
		LS6		0.85	0.85	0.87	0.87	0.89	0.84	0.88	0.79	0.80	0.72	0.73	0.70	0.73	0.73
		LS7		0.81	0.82	0.83	0.81	0.84	0.76	0.81	0.71	0.73	0.64	0.65	0.65	0.67	0.70
		LS8		0.80	0.82	0.81	0.78	0.83	0.75	0.82	0.71	0.73	0.65	0.64	0.66	0.65	0.69
		LS9		0.79	0.81	0.81	0.78	0.84	0.77	0.88	0.76	0.78	0.68	0.66	0.66	0.62	0.68
		LS10		0.81	0.79	0.83	0.83	0.87	0.80	0.88	0.77	0.79	0.69	0.68	0.66	0.66	0.65
		LS11		0.82	0.78	0.86	0.86	0.88	0.82	0.87	0.76	0.78	0.68	0.69	0.66	0.69	0.66
		LS12		0.90	0.91	0.99	1.00	0.99	0.98	1.00	0.95	0.96	0.89	0.92	0.84	0.91	0.81
		LS13		0.86	0.87	0.96	0.96	0.96	0.94	0.96	0.91	0.92	0.85	0.87	0.79	0.86	0.78
		LS14		0.85	0.85	0.96	0.96	0.98	0.93	0.97	0.90	0.91	0.84	0.86	0.79	0.83	0.77
		LS15		0.84	0.83	0.95	0.95	0.97	0.91	0.96	0.87	0.88	0.80	0.82	0.76	0.79	0.75
		LS16		0.85	0.81	0.94	0.94	0.96	0.90	0.94	0.84	0.85	0.75	0.77	0.72	0.76	0.72
		LS18		0.79	0.79	0.80	0.80	0.82	0.82	0.89	0.87	0.88	0.84	0.81	0.76	0.79	0.74
		LS19		0.79	0.79	0.82	0.81	0.91	0.87	0.88	0.82	0.86	0.79	0.78	0.75	0.72	0.70
		LS20		0.82	0.82	0.84	0.83	0.91	0.86	0.89	0.80	0.83	0.74	0.73	0.72	0.70	0.71
		LS21		0.82	0.82	0.84	0.83	0.91	0.86	0.89	0.80	0.83	0.74	0.73	0.72	0.70	0.71
		LS22		0.85	0.85	0.87	0.87	0.90	0.89	0.97	0.95	0.96	0.92	0.89	0.85	0.88	0.83
		LS23		0.81	0.81	0.89	0.89	0.99	0.94	0.96	0.89	0.93	0.86	0.85	0.81	0.78	0.74
		LS24		0.82	0.80	0.90	0.90	0.98	0.92	0.95	0.86	0.89	0.80	0.79	0.75	0.74	0.71
		LS25		0.82	0.80	0.90	0.90	0.98	0.92	0.95	0.86	0.89	0.80	0.79	0.75	0.74	0.71
		LS26		0.94	0.95	0.96	0.97	0.96	0.95	0.97	0.93	0.94	0.88	0.90	0.82	0.89	0.79
		LS27		0.91	0.92	0.93	0.94	0.92	0.91	0.93	0.89	0.90	0.84	0.86	0.79	0.86	0.77
		LS28		0.85	0.86	0.88	0.88	0.86	0.85	0.87	0.84	0.85	0.79	0.81	0.75	0.82	0.74
		LS29		0.85	0.86	0.87	0.87	0.88	0.87	0.91	0.88	0.89	0.84	0.80	0.74	0.78	0.73
		LS30		0.88	0.88	0.94	0.94	0.95	0.93	0.98	0.94	0.95	0.90	0.86	0.80	0.84	0.79
		LS31		0.86	0.87	0.92	0.93	0.91	0.90	0.92	0.88	0.89	0.84	0.86	0.80	0.87	0.80
		LS32		0.85	0.86	0.92	0.92	0.91	0.90	0.92	0.88	0.89	0.84	0.88	0.81	0.86	0.78
		LS33		0.88	0.89	0.95	0.95	0.95	0.94	0.96	0.92	0.93	0.87	0.91	0.84	0.89	0.80
	Longitudinal Stiffener Stress Points side -	LS2		0.92	0.93	0.90	0.92	0.91	0.92	0.93	0.92	0.93	0.89	0.92	0.85	0.92	0.83
		LS3		0.87	0.88	0.85	0.87	0.86	0.87	0.88	0.87	0.88	0.85	0.88	0.82	0.88	0.82
		LS4		0.85	0.86	0.82	0.85	0.85	0.84	0.88	0.86	0.87	0.85	0.87	0.82	0.86	0.83
		LS5		0.82	0.83	0.79	0.82	0.82	0.81	0.84	0.81	0.82	0.81	0.82	0.80	0.83	0.81
		LS6		0.79	0.80	0.76	0.78	0.79	0.78	0.81	0.78	0.79	0.76	0.78	0.78	0.80	0.82
		LS7		0.70	0.72	0.66	0.68	0.69	0.68	0.73	0.69	0.70	0.68	0.69	0.75	0.75	0.84
		LS8		0.72	0.73	0.62	0.64	0.67	0.70	0.78	0.71	0.73	0.72	0.70	0.80	0.76	0.85
		LS9		0.76	0.76	0.63	0.65	0.70	0.73	0.84	0.78	0.80	0.81	0.79	0.86	0.80	0.87
		LS10		0.79	0.80	0.67	0.70	0.73	0.73	0.80	0.79	0.81	0.82	0.80	0.87	0.84	0.90
		LS11		0.82	0.82	0.70	0.73	0.75	0.75	0.79	0.78	0.79	0.80	0.80	0.86	0.87	0.92
		LS12		0.94	0.95	0.91	0.93	0.93	0.95	0.96	0.96	0.97	0.95	0.97	0.94	1.00	0.94
		LS13		0.90	0.92	0.86	0.88	0.88	0.90	0.91	0.92	0.93	0.92	0.94	0.91	0.97	0.94
		LS14		0.91	0.92	0.84	0.87	0.87	0.88	0.92	0.91	0.92	0.93	0.94	0.93	0.97	0.96
		LS15		0.89	0.90	0.81	0.84	0.85	0.86	0.89	0.88	0.89	0.89	0.91	0.93	0.95	0.96
		LS16		0.88	0.89	0.78	0.81	0.82	0.83	0.87	0.85	0.86	0.86	0.88	0.92	0.93	0.96
		LS18		0.79	0.79	0.78	0.78	0.81	0.80	0.87	0.86	0.87	0.84	0.82	0.78	0.80	0.76
		LS19		0.77	0.77	0.73	0.75	0.83	0.82	0.83	0.81	0.85	0.83	0.81	0.80	0.77	0.76
		LS20		0.76	0.76	0.72	0.74	0.80	0.79	0.81	0.78	0.81	0.78	0.78	0.78	0.76	0.80
		LS21		0.76	0.76	0.72	0.74	0.80	0.79	0.81	0.78	0.81	0.78	0.78	0.78	0.76	0.80
		LS22		0.86	0.86	0.85	0.85	0.88	0.88	0.96	0.95	0.96	0.94	0.91	0.88	0.91	0.87
		LS23		0.86	0.86	0.79	0.81	0.89	0.90	0.91	0.90	0.94	0.94	0.92	0.94	0.89	0.90
		LS24		0.86	0.86	0.76	0.79	0.86	0.86	0.89	0.87	0.90	0.90	0.88	0.92	0.89	0.92
		LS25		0.86	0.86	0.76	0.79	0.86	0.86	0.89	0.87	0.90	0.90	0.88	0.92	0.89	0.92
		LS26		0.93	0.94	0.92	0.94	0.92	0.93	0.94	0.93	0.94	0.90	0.92	0.85	0.92	0.83
		LS27		0.90	0.91	0.88	0.90	0.88	0.89	0.90	0.89	0.90	0.86	0.88	0.82	0.89	0.80
		LS28		0.84	0.85	0.83	0.85	0.82	0.83	0.84	0.83	0.84	0.81	0.83	0.77	0.84	0.78
		LS29		0.84	0.85	0.82	0.84	0.84	0.85	0.88	0.87	0.88	0.86	0.82	0.77	0.81	0.76
		LS30		0.91	0.91	0.87	0.89	0.90	0.91	0.95	0.95	0.96	0.94	0.90	0.88	0.91	0.88
		LS31		0.89	0.90	0.87	0.88	0.86	0.87	0.89	0.89	0.90	0.88	0.89	0.86	0.93	0.88
		LS32		0.88	0.89	0.86	0.88	0.87	0.88	0.89	0.89	0.90	0.88	0.91	0.88	0.92	0.86
		LS33		0.90	0.91	0.89	0.91	0.91	0.92	0.93	0.92	0.93	0.91	0.95	0.91	0.95	0.88
		MAX		**0.940**	**0.952**	**0.991**	**0.995**	**0.995**	**0.980**	**0.996**	**0.958**	**0.968**	**0.951**	**0.972**	**0.939**	**0.997**	**0.962**
	Governing Stiffener			**LS26**	**LS26**	**LS12**	**LS12**	**LS12**	**LS12**	**LS12**	**LS12**	**LS12**	**LS12**	**LS12**	**LS23**	**LS12**	**LS15**
	side			**+**	**+**	**+**	**+**	**+**	**+**	**+**	**-**	**-**	**-**	**-**	**-**	**-**	**-**
	reduction factor for governing			**0.952**	**0.952**	**0.982**	**0.982**	**0.981**	**0.981**	**0.980**	**0.980**	**0.980**	**0.980**	**0.997**	**0.851**	**0.978**	**0.955**
	Governing Case for Max Stiffener			**min L001**	**min L001**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**
		Ns		**-1642**	**-1634**	**-1918**	**-1902**	**-1902**	**-1879**	**-1879**	**-1856**	**-1856**	**-1833**	**-1833**	**-1813**	**-1812**	**-1795**
		My		**4806**	**5008**	**-3865**	**-4158**	**-4158**	**-4389**	**-4389**	**-4293**	**-4294**	**-3954**	**-3954**	**-3399**	**-3400**	**-2599**
		Mz		**105**	**111**	**672**	**520**	**520**	**261**	**261**	**-46**	**-46**	**-379**	**-379**	**-680**	**-680**	**-954**
Summary	Plates	UR		**0.94**	**0.95**	**0.99**	**1.00**	**0.99**	**0.98**	**1.00**	**0.96**	**0.97**	**0.95**	**0.99**	**0.95**	**0.99**	**0.92**
	Stiffeners	UR		**0.94**	**0.95**	**0.99**	**1.00**	**0.99**	**0.98**	**1.00**	**0.96**	**0.97**	**0.95**	**0.97**	**0.94**	**1.00**	**0.96**
	Overall Max	UR		**0.94**	**0.95**	**0.99**	**1.00**	**0.99**	**0.98**	**1.00**	**0.96**	**0.97**	**0.95**	**0.99**	**0.95**	**1.00**	**0.96**

General	Analysis Section			43	44	45	46	47	48
	S-coordinate			357.4	369.9	369.9	374.447	374.447	383.619
	Cross Section/Segment		Units	20	20	21	21	21	22
Section Properties	Gross Section	A	(mm^2)	7.43E+06	7.43E+06	7.71E+06	7.71E+06	7.71E+06	5.53E+06
		Y	(mm)	0.0	0.0	0.0	0.0	0.0	0.0
		I_{1_Y}	(mm^4)	3.02E+14	3.02E+14	3.13E+14	3.13E+14	3.13E+14	1.93E+14
		I_{2_Z}	(mm^4)	1.16E+14	1.16E+14	1.19E+14	1.19E+14	1.19E+14	7.77E+13
VERIFICATION - UTILIZATION RATIOS									
	Plate Stress Points for Corners	SP1		0.82	0.73	0.68	0.65	0.66	0.87
		SP2		0.76	0.69	0.67	0.65	0.62	0.80
		SP3		0.67	0.68	0.66	0.67	0.53	0.65
		SP4		0.65	0.64	0.62	0.64	0.57	0.71
		SP5		0.78	0.72	0.69	0.67	0.67	0.88
		SP6		0.88	0.80	0.75	0.72	0.72	0.95
		SP7		0.80	0.74	0.72	0.70	0.72	0.90
		SP8		0.77	0.82	0.80	0.82	0.80	0.89
		SP9		0.88	0.90	0.87	0.87	0.87	0.97
		SP10		0.89	0.84	0.81	0.80	0.80	0.99
		SP11		0.61	0.59	0.57	0.56	0.53	0.70
		SP12		0.67	0.64	0.61	0.60	0.59	0.78
		SP13		0.65	0.64	0.62	0.63	0.63	0.78
		SP14		0.76	0.75	0.72	0.72	0.71	0.88
		Max		**0.89**	**0.90**	**0.87**	**0.87**	**0.87**	**0.99**
	Governing Stress Point			**SP10**	**SP9**	**SP9**	**SP9**	**SP9**	**SP10**
	reduction factor			**1.00**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
	Govern. Case for Max Stress Point			**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**
		Ns		**-1795**	**-1782**	**-1780**	**-1776**	**-1745**	**-1735**
		My		**-2599**	**-1888**	**-1888**	**-1595**	**-1753**	**-1103**
		Mz		**-953**	**-1147**	**-1147**	**-1223**	**-1234**	**-700**
	Longitudinal Stiffener Stress Points side +	LS2		0.76	0.70	0.67	0.64	0.62	0.00
		LS3		0.73	0.68	0.65	0.63	0.60	0.00
		LS4		0.72	0.68	0.65	0.64	0.59	0.00
		LS5		0.70	0.69	0.66	0.66	0.57	0.00
		LS6		0.69	0.70	0.67	0.67	0.55	0.00
		LS7		0.66	0.69	0.66	0.67	0.53	0.65
		LS8		0.65	0.69	0.66	0.67	0.52	0.65
		LS9		0.64	0.69	0.66	0.67	0.52	0.67
		LS10		0.62	0.67	0.64	0.66	0.54	0.69
		LS11		0.63	0.66	0.64	0.66	0.55	0.70
		LS12		0.78	0.72	0.69	0.66	0.66	0.00
		LS13		0.75	0.69	0.66	0.64	0.64	0.00
		LS14		0.74	0.69	0.66	0.64	0.63	0.00
		LS15		0.71	0.67	0.64	0.63	0.61	0.00
		LS16		0.68	0.65	0.62	0.65	0.59	0.00
		LS18		0.68	0.65	0.59	0.58	0.57	0.77
		LS19		0.66	0.66	0.62	0.63	0.56	0.71
		LS20		0.67	0.68	0.65	0.65	0.55	0.68
		LS21		0.67	0.68	0.65	0.65	0.55	0.68
		LS22		0.76	0.73	0.66	0.65	0.64	0.86
		LS23		0.70	0.66	0.63	0.62	0.61	0.79
		LS24		0.68	0.64	0.61	0.63	0.59	0.75
		LS25		0.68	0.64	0.61	0.63	0.59	0.75
		LS26		0.76	0.69	0.66	0.63	0.62	0.81
		LS27		0.73	0.68	0.64	0.62	0.60	0.79
		LS28		0.70	0.66	0.62	0.60	0.58	0.77
		LS29		0.70	0.66	0.61	0.59	0.57	0.75
		LS30		0.75	0.71	0.65	0.64	0.64	0.84
		LS31		0.75	0.70	0.66	0.64	0.64	0.85
		LS32		0.75	0.69	0.67	0.65	0.65	0.86
		LS33		0.77	0.71	0.68	0.66	0.66	0.87
	Longitudinal Stiffener Stress Points side -	LS2		0.81	0.76	0.72	0.71	0.73	0.00
		LS3		0.79	0.75	0.72	0.71	0.74	0.00
		LS4		0.80	0.77	0.74	0.74	0.76	0.00
		LS5		0.78	0.80	0.77	0.78	0.77	0.00
		LS6		0.78	0.83	0.80	0.82	0.81	0.00
		LS7		0.79	0.85	0.81	0.83	0.83	0.91
		LS8		0.80	0.86	0.82	0.84	0.84	0.92
		LS9		0.82	0.87	0.83	0.85	0.85	0.93
		LS10		0.85	0.88	0.84	0.85	0.85	0.95
		LS11		0.87	0.89	0.85	0.86	0.86	0.96
		LS12		0.90	0.87	0.83	0.81	0.81	0.00
		LS13		0.90	0.87	0.83	0.82	0.82	0.00
		LS14		0.92	0.90	0.86	0.86	0.86	0.00
		LS15		0.92	0.91	0.87	0.87	0.87	0.00
		LS16		0.92	0.92	0.88	0.89	0.88	0.00
		LS18		0.69	0.67	0.60	0.59	0.60	0.80
		LS19		0.71	0.74	0.71	0.72	0.72	0.85
		LS20		0.76	0.80	0.77	0.78	0.78	0.88
		LS21		0.76	0.80	0.77	0.78	0.78	0.88
		LS22		0.80	0.77	0.70	0.69	0.69	0.90
		LS23		0.84	0.84	0.80	0.80	0.79	0.95
		LS24		0.87	0.88	0.84	0.84	0.84	0.97
		LS25		0.87	0.88	0.84	0.84	0.84	0.97
		LS26		0.79	0.73	0.69	0.67	0.69	0.88
		LS27		0.77	0.72	0.68	0.66	0.68	0.86
		LS28		0.74	0.69	0.65	0.64	0.66	0.84
		LS29		0.73	0.70	0.64	0.63	0.65	0.83
		LS30		0.84	0.82	0.75	0.74	0.74	0.93
		LS31		0.83	0.80	0.76	0.75	0.74	0.94
		LS32		0.82	0.79	0.76	0.75	0.75	0.94
		LS33		0.85	0.80	0.78	0.76	0.76	0.96
		MAX		**0.917**	**0.923**	**0.884**	**0.885**	**0.884**	**0.967**
	Governing Stiffener			**LS14**	**LS16**	**LS16**	**LS16**	**LS16**	**LS24**
		side		**-**	**-**	**-**	**-**	**-**	**-**
	reduction factor for governing			**0.966**	**0.966**	**0.975**	**0.975**	**0.975**	**0.964**
	Governing Case for Max Stiffener			**min NS**	**min NS**	**min NS**	**min NS**	**min NS**	**min NS**
		Ns		**-1795**	**-1782**	**-1780**	**-1776**	**-1745**	**-1735**
		My		**-2599**	**-1888**	**-1888**	**-1595**	**-1753**	**-1103**
		Mz		**-953**	**-1147**	**-1147**	**-1223**	**-1234**	**-700**
Summary	**Plates**	**UR**		**0.89**	**0.90**	**0.87**	**0.87**	**0.87**	**0.99**
	Stiffeners	**UR**		**0.92**	**0.92**	**0.88**	**0.89**	**0.88**	**0.97**
	Overall Max	**UR**		**0.92**	**0.92**	**0.88**	**0.89**	**0.88**	**0.99**

The following figure shows a plot of the utilization ratios for all stress points over the height of the tower. From the plot it is clear that for the ULS 7 load combination being considered, all utilization ratios are below 1.0 for the Sicilia tower leg 1.



5.1.1.7 Verification for All Load Combinations

Using the same procedures as illustrated in the example calculations above, all relevant load combinations were checked for both tower legs and both the Sicilia and Calabria towers. Due to the size of the data produced by these calculations, only the governing utilization ratios are provided here.

Sicilia Tower (1/4)

Analysis Section		1	2	3	4	5	6	7	8	9	10	11	12
Tower Segment		1	1	2	2	3	3	4	4	5	5	6	6
Elevation		18	28	28	40	40	55	55	71	71	87	87	105
Load Combin. Description	**Leg**	**Governing Utilization Ratio**											
ULS 7: Seismic, Longit.	Leg 1	1.00	0.93	1.00	0.93	1.00	0.93	1.00	0.96	1.00	0.96	1.00	0.99
ULS 7: Seismic, Trans.	Leg 1	0.93	0.87	0.94	0.88	0.94	0.90	0.97	0.94	0.98	0.94	0.97	0.96
SILS 2: Seismic, Longit.	Leg 1	0.92	0.86	0.92	0.85	0.91	0.81	0.87	0.77	0.82	0.78	0.83	0.80
SILS 2: Seismic, Trans.	Leg 1	0.84	0.79	0.84	0.78	0.84	0.76	0.81	0.75	0.79	0.75	0.78	0.76
ULS 2 & 6 Wind, North	Leg 1	0.70	0.65	0.72	0.68	0.74	0.71	0.78	0.76	0.79	0.78	0.81	0.80
ULS 2 & 6 Wind, West	Leg 1	0.62	0.60	0.68	0.66	0.75	0.74	0.81	0.79	0.82	0.80	0.83	0.83
ULS 2 & 6 Wind, South	Leg 1	0.84	0.81	0.92	0.86	0.93	0.85	0.93	0.89	0.96	0.91	0.93	0.95
ULS 2 & 6 Wind, East	Leg 1	0.57	0.56	0.65	0.64	0.72	0.72	0.78	0.77	0.80	0.79	0.81	0.82
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.65	0.62	0.71	0.67	0.74	0.72	0.79	0.76	0.80	0.78	0.81	0.79
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.78	0.75	0.85	0.81	0.88	0.82	0.89	0.87	0.91	0.88	0.91	0.92
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.65	0.61	0.69	0.64	0.72	0.71	0.78	0.76	0.79	0.78	0.81	0.80
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.75	0.72	0.82	0.78	0.85	0.79	0.86	0.84	0.89	0.86	0.89	0.91
SILS 1: Wind, North	Leg 1	0.64	0.58	0.65	0.58	0.63	0.57	0.62	0.61	0.64	0.63	0.65	0.65
SILS 1: Wind, West	Leg 1	0.48	0.48	0.53	0.52	0.58	0.58	0.63	0.62	0.65	0.64	0.66	0.67
SILS 1: Wind, South	Leg 1	0.78	0.73	0.83	0.77	0.84	0.76	0.83	0.76	0.83	0.78	0.80	0.82
SILS 1: Wind, East	Leg 1	0.54	0.52	0.56	0.55	0.61	0.60	0.65	0.64	0.67	0.65	0.67	0.68
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.56	0.52	0.58	0.54	0.59	0.57	0.62	0.61	0.64	0.63	0.65	0.65
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.68	0.64	0.73	0.68	0.74	0.68	0.74	0.71	0.76	0.73	0.75	0.77
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.59	0.55	0.62	0.56	0.61	0.57	0.62	0.61	0.64	0.63	0.65	0.65
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.70	0.66	0.76	0.71	0.77	0.70	0.77	0.73	0.78	0.74	0.76	0.77
ULS 5: Dead & Live	Leg 1	0.60	0.60	0.68	0.68	0.77	0.76	0.83	0.81	0.85	0.83	0.86	0.86
ULS 7: Seismic, Longit.	Leg 2	0.99	0.93	0.99	0.93	1.00	0.92	0.98	0.93	0.98	0.95	0.99	0.98
ULS 7: Seismic, Trans.	Leg 2	0.91	0.86	0.93	0.87	0.93	0.87	0.94	0.90	0.94	0.92	0.96	0.95
SILS 2: Seismic, Longit.	Leg 2	0.91	0.85	0.91	0.84	0.90	0.80	0.86	0.75	0.81	0.77	0.82	0.79
SILS 2: Seismic, Trans.	Leg 2	0.82	0.78	0.83	0.77	0.82	0.75	0.80	0.72	0.77	0.74	0.77	0.77
ULS 2 & 6 Wind, North	Leg 2	0.81	0.77	0.87	0.83	0.91	0.86	0.94	0.89	0.97	0.90	0.94	0.94
ULS 2 & 6 Wind, West	Leg 2	0.60	0.59	0.66	0.65	0.74	0.73	0.80	0.78	0.82	0.80	0.83	0.82
ULS 2 & 6 Wind, South	Leg 2	0.71	0.67	0.76	0.71	0.78	0.71	0.78	0.76	0.80	0.78	0.81	0.81
ULS 2 & 6 Wind, East	Leg 2	0.57	0.56	0.65	0.64	0.72	0.71	0.78	0.76	0.79	0.79	0.81	0.81
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.74	0.72	0.82	0.78	0.84	0.82	0.88	0.86	0.90	0.88	0.91	0.91
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.68	0.65	0.74	0.70	0.76	0.72	0.78	0.76	0.80	0.78	0.80	0.80
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.73	0.71	0.81	0.78	0.84	0.80	0.87	0.84	0.90	0.86	0.89	0.90
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.65	0.62	0.70	0.67	0.73	0.71	0.78	0.76	0.80	0.78	0.80	0.80
SILS 1: Wind, North	Leg 2	0.75	0.71	0.81	0.77	0.84	0.77	0.84	0.77	0.85	0.77	0.80	0.80
SILS 1: Wind, West	Leg 2	0.48	0.48	0.53	0.52	0.58	0.58	0.63	0.62	0.65	0.64	0.67	0.67
SILS 1: Wind, South	Leg 2	0.62	0.58	0.64	0.59	0.65	0.57	0.64	0.61	0.64	0.63	0.66	0.66
SILS 1: Wind, East	Leg 2	0.54	0.52	0.56	0.55	0.61	0.60	0.65	0.64	0.67	0.65	0.68	0.67
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.65	0.62	0.70	0.67	0.73	0.69	0.75	0.71	0.77	0.72	0.75	0.76
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.56	0.53	0.60	0.56	0.61	0.57	0.62	0.61	0.64	0.63	0.66	0.65
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.68	0.65	0.74	0.70	0.76	0.71	0.78	0.73	0.78	0.73	0.77	0.76
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.58	0.55	0.62	0.57	0.62	0.57	0.62	0.61	0.64	0.63	0.65	0.65
ULS 5: Dead & Live	Leg 2	0.60	0.60	0.68	0.67	0.77	0.76	0.83	0.81	0.85	0.83	0.86	0.85
Max of All Load Combinations (either leg)		**1.00**	**0.93**	**1.00**	**0.93**	**1.00**	**0.93**	**1.00**	**0.96**	**1.00**	**0.96**	**1.00**	**0.99**

Sicilia Tower (2/4)

Analysis Section	13	14	15	16	17	18	19	20	21	22	23	24
Tower Segment	7	7	8	8	8	8	9	9	10	10	11	11
Elevation	105	123.65	123.65	124	124	143	143	163	163	183	183	203

Load Combin. Description	Leg	Governing Utilization Ratio											
ULS 7: Seismic, Longit.	Leg 1	0.99	0.98	0.99	0.99	0.99	0.95	1.00	0.98	0.99	0.99	1.00	1.00
ULS 7: Seismic, Trans.	Leg 1	0.94	0.94	0.95	0.95	0.95	0.91	0.96	0.94	0.95	0.95	0.96	0.96
SILS 2: Seismic, Longit.	Leg 1	0.82	0.81	0.81	0.81	0.81	0.84	0.87	0.88	0.89	0.89	0.90	0.91
SILS 2: Seismic, Trans.	Leg 1	0.79	0.78	0.77	0.77	0.77	0.79	0.82	0.83	0.83	0.84	0.84	0.86
ULS 2 & 6 Wind, North	Leg 1	0.76	0.77	0.83	0.83	0.83	0.75	0.80	0.75	0.80	0.80	0.81	0.81
ULS 2 & 6 Wind, West	Leg 1	0.78	0.78	0.75	0.75	0.75	0.73	0.78	0.76	0.81	0.81	0.82	0.83
ULS 2 & 6 Wind, South	Leg 1	0.89	0.95	0.997	1.00	1.00	0.92	0.99	0.89	0.96	0.89	0.90	0.93
ULS 2 & 6 Wind, East	Leg 1	0.77	0.78	0.75	0.75	0.75	0.74	0.79	0.77	0.82	0.82	0.83	0.83
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.76	0.76	0.78	0.78	0.78	0.72	0.77	0.75	0.80	0.80	0.81	0.81
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.86	0.89	0.91	0.91	0.91	0.85	0.92	0.83	0.90	0.87	0.88	0.89
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.76	0.76	0.78	0.77	0.77	0.72	0.78	0.75	0.80	0.80	0.81	0.82
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.85	0.89	0.91	0.91	0.91	0.86	0.92	0.85	0.91	0.88	0.89	0.89
SILS 1: Wind, North	Leg 1	0.62	0.63	0.75	0.74	0.74	0.64	0.70	0.61	0.65	0.65	0.66	0.66
SILS 1: Wind, West	Leg 1	0.64	0.65	0.63	0.63	0.63	0.62	0.66	0.65	0.68	0.67	0.68	0.69
SILS 1: Wind, South	Leg 1	0.77	0.85	0.90	0.90	0.90	0.82	0.88	0.77	0.84	0.75	0.76	0.79
SILS 1: Wind, East	Leg 1	0.64	0.65	0.62	0.62	0.62	0.60	0.65	0.62	0.66	0.65	0.66	0.66
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.62	0.63	0.69	0.69	0.69	0.62	0.67	0.61	0.65	0.65	0.66	0.66
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.72	0.78	0.81	0.81	0.81	0.75	0.81	0.73	0.78	0.74	0.75	0.76
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.62	0.63	0.69	0.68	0.68	0.61	0.65	0.61	0.65	0.65	0.66	0.66
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.73	0.77	0.80	0.80	0.80	0.74	0.79	0.71	0.77	0.71	0.72	0.74
ULS 5: Dead & Live	Leg 1	0.82	0.82	0.79	0.79	0.79	0.77	0.83	0.81	0.86	0.85	0.86	0.86
ULS 7: Seismic, Longit.	Leg 2	0.99	0.99	0.98	0.98	0.98	0.94	0.99	0.98	0.96	0.97	0.97	0.99
ULS 7: Seismic, Trans.	Leg 2	0.94	0.96	0.93	0.93	0.93	0.89	0.94	0.93	0.92	0.93	0.93	0.94
SILS 2: Seismic, Longit.	Leg 2	0.82	0.80	0.81	0.81	0.81	0.82	0.86	0.87	0.88	0.89	0.90	0.91
SILS 2: Seismic, Trans.	Leg 2	0.77	0.78	0.77	0.77	0.77	0.78	0.81	0.82	0.83	0.84	0.84	0.86
ULS 2 & 6 Wind, North	Leg 2	0.89	0.95	0.96	0.96	0.96	0.88	0.95	0.87	0.94	0.89	0.90	0.93
ULS 2 & 6 Wind, West	Leg 2	0.78	0.79	0.73	0.73	0.73	0.72	0.77	0.76	0.81	0.81	0.82	0.83
ULS 2 & 6 Wind, South	Leg 2	0.77	0.78	0.88	0.88	0.88	0.78	0.85	0.76	0.80	0.80	0.81	0.81
ULS 2 & 6 Wind, East	Leg 2	0.77	0.79	0.73	0.73	0.73	0.73	0.78	0.77	0.82	0.81	0.83	0.83
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.86	0.89	0.88	0.88	0.88	0.82	0.89	0.82	0.89	0.87	0.88	0.89
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.76	0.77	0.82	0.82	0.82	0.75	0.81	0.75	0.80	0.80	0.81	0.81
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.85	0.89	0.88	0.88	0.88	0.82	0.88	0.84	0.88	0.87	0.88	0.89
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.76	0.77	0.82	0.82	0.82	0.76	0.81	0.76	0.80	0.80	0.81	0.82
SILS 1: Wind, North	Leg 2	0.76	0.84	0.87	0.87	0.87	0.79	0.86	0.76	0.82	0.75	0.76	0.78
SILS 1: Wind, West	Leg 2	0.64	0.65	0.63	0.63	0.63	0.62	0.66	0.64	0.68	0.67	0.68	0.69
SILS 1: Wind, South	Leg 2	0.63	0.64	0.76	0.76	0.76	0.66	0.71	0.62	0.66	0.65	0.66	0.66
SILS 1: Wind, East	Leg 2	0.64	0.65	0.62	0.62	0.62	0.60	0.64	0.62	0.66	0.65	0.66	0.66
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.72	0.77	0.79	0.79	0.79	0.73	0.79	0.72	0.77	0.74	0.75	0.75
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.62	0.63	0.70	0.70	0.70	0.63	0.68	0.62	0.65	0.65	0.66	0.66
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.72	0.77	0.78	0.78	0.78	0.72	0.77	0.70	0.75	0.71	0.72	0.73
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.62	0.63	0.70	0.69	0.69	0.62	0.67	0.62	0.65	0.65	0.66	0.66
ULS 5: Dead & Live	Leg 2	0.82	0.83	0.78	0.78	0.78	0.77	0.83	0.81	0.85	0.85	0.86	0.86
Max of All Load Combinations (either leg)		**0.99**	**0.99**	**1.00**	**1.00**	**1.00**	**0.95**	**1.00**	**0.98**	**0.99**	**0.99**	**1.00**	**1.00**

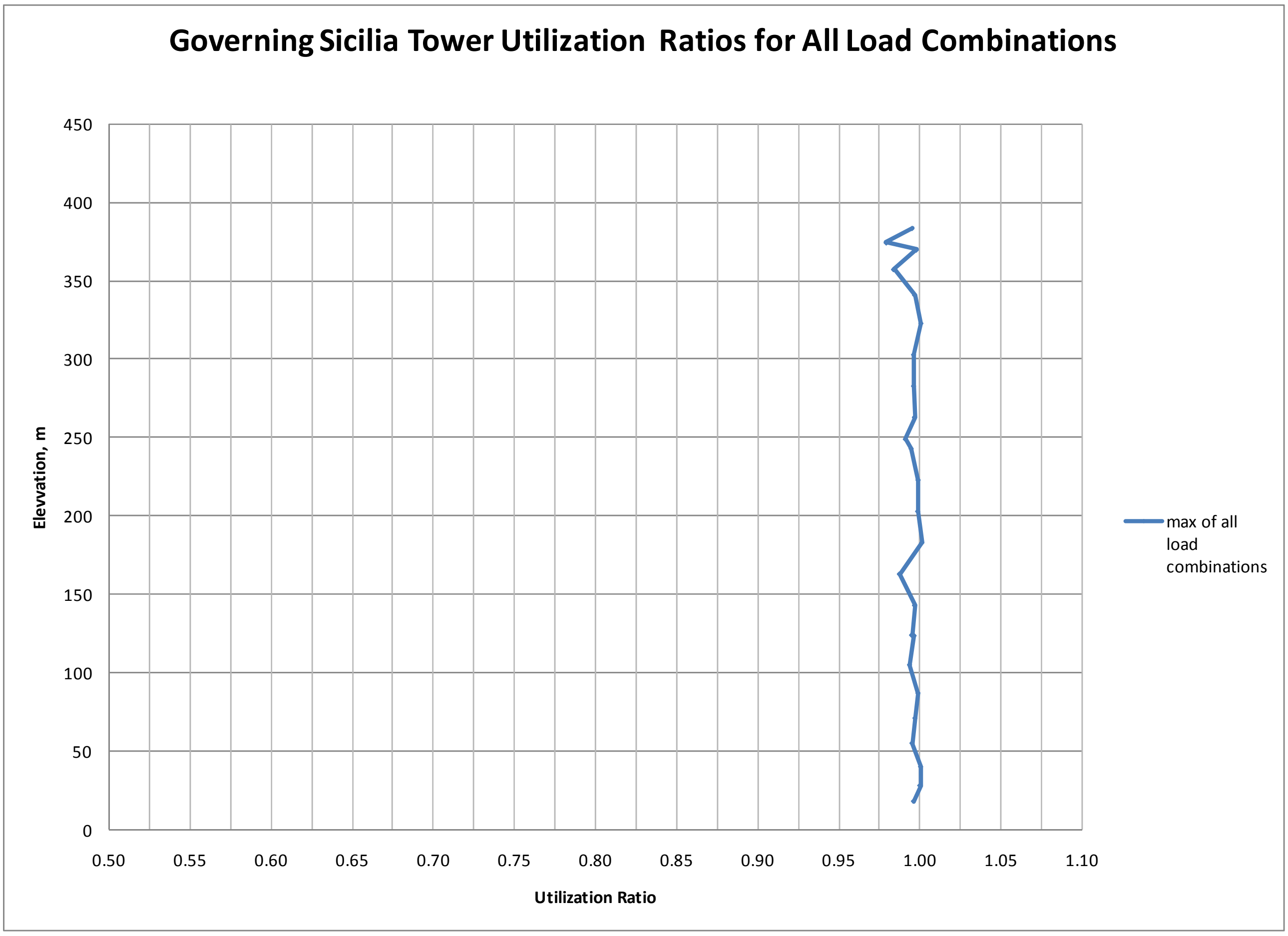
Sicilia Tower (3/4)

Load Combin. Description	Leg	25	26	27	28	29	30	31	32	33	34	35	36
Analysis Section		25	26	27	28	29	30	31	32	33	34	35	36
Tower Segment		12	12	13	13	14	14	14	14	15	15	16	16
Elevation		203	223	223	243	243	249.048	249.048	263	263	283	283	303
Load Combin. Description	**Leg**	**Governing Utilization Ratio**											
ULS 7: Seismic, Longit.	Leg 1	0.98	1.00	0.95	0.99	0.94	0.95	0.99	1.00	0.99	0.98	1.00	0.96
ULS 7: Seismic, Trans.	Leg 1	0.95	0.96	0.92	0.95	0.90	0.90	0.92	0.94	0.96	0.92	0.94	0.91
SILS 2: Seismic, Longit.	Leg 1	0.90	0.93	0.89	0.95	0.89	0.91	0.91	0.93	0.91	0.92	0.92	0.89
SILS 2: Seismic, Trans.	Leg 1	0.84	0.87	0.83	0.87	0.82	0.83	0.83	0.84	0.83	0.83	0.84	0.82
ULS 2 & 6 Wind, North	Leg 1	0.78	0.83	0.78	0.85	0.81	0.83	0.77	0.74	0.80	0.73	0.81	0.73
ULS 2 & 6 Wind, West	Leg 1	0.80	0.81	0.75	0.76	0.72	0.72	0.73	0.72	0.78	0.74	0.78	0.75
ULS 2 & 6 Wind, South	Leg 1	0.88	0.96	0.90	0.99	0.95	0.97	0.92	0.87	0.93	0.81	0.92	0.78
ULS 2 & 6 Wind, East	Leg 1	0.80	0.80	0.74	0.74	0.71	0.70	0.73	0.72	0.78	0.73	0.78	0.75
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.78	0.80	0.75	0.81	0.77	0.78	0.70	0.69	0.75	0.73	0.79	0.75
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.87	0.91	0.86	0.92	0.89	0.90	0.86	0.83	0.89	0.80	0.87	0.78
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.79	0.82	0.78	0.82	0.79	0.80	0.73	0.72	0.77	0.73	0.79	0.74
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.87	0.89	0.84	0.90	0.87	0.88	0.87	0.83	0.89	0.80	0.87	0.78
SILS 1: Wind, North	Leg 1	0.64	0.68	0.64	0.71	0.69	0.70	0.65	0.61	0.65	0.60	0.67	0.58
SILS 1: Wind, West	Leg 1	0.67	0.68	0.64	0.64	0.61	0.61	0.60	0.59	0.64	0.60	0.62	0.60
SILS 1: Wind, South	Leg 1	0.75	0.85	0.80	0.89	0.86	0.88	0.80	0.75	0.80	0.69	0.76	0.63
SILS 1: Wind, East	Leg 1	0.64	0.64	0.59	0.59	0.57	0.57	0.57	0.56	0.61	0.57	0.61	0.58
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.64	0.67	0.64	0.69	0.67	0.67	0.61	0.59	0.62	0.59	0.64	0.60
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.74	0.80	0.76	0.82	0.79	0.80	0.75	0.71	0.75	0.66	0.71	0.64
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.64	0.65	0.61	0.67	0.64	0.65	0.59	0.57	0.61	0.57	0.64	0.58
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.70	0.76	0.72	0.78	0.75	0.76	0.72	0.68	0.73	0.64	0.71	0.61
ULS 5: Dead & Live	Leg 1	0.84	0.84	0.78	0.77	0.73	0.73	0.74	0.73	0.80	0.76	0.81	0.78
ULS 7: Seismic, Longit.	Leg 2	0.97	0.99	0.95	0.99	0.93	0.95	0.94	0.97	0.96	0.97	0.98	0.99
ULS 7: Seismic, Trans.	Leg 2	0.92	0.95	0.91	0.94	0.89	0.90	0.90	0.90	0.95	0.90	0.93	0.89
SILS 2: Seismic, Longit.	Leg 2	0.90	0.92	0.88	0.96	0.90	0.91	0.91	0.93	0.92	0.92	0.93	0.91
SILS 2: Seismic, Trans.	Leg 2	0.84	0.86	0.82	0.88	0.83	0.84	0.83	0.85	0.84	0.83	0.84	0.81
ULS 2 & 6 Wind, North	Leg 2	0.88	0.95	0.90	0.98	0.95	0.97	0.90	0.85	0.92	0.81	0.91	0.77
ULS 2 & 6 Wind, West	Leg 2	0.80	0.81	0.75	0.76	0.72	0.72	0.73	0.72	0.78	0.74	0.78	0.75
ULS 2 & 6 Wind, South	Leg 2	0.79	0.81	0.77	0.83	0.80	0.78	0.84	0.79	0.85	0.75	0.85	0.74
ULS 2 & 6 Wind, East	Leg 2	0.80	0.81	0.75	0.76	0.72	0.72	0.71	0.70	0.77	0.72	0.78	0.75
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.86	0.90	0.85	0.91	0.88	0.89	0.85	0.82	0.88	0.79	0.86	0.78
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.79	0.80	0.75	0.81	0.78	0.74	0.77	0.75	0.80	0.74	0.82	0.75
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.87	0.91	0.86	0.92	0.88	0.89	0.84	0.81	0.87	0.78	0.86	0.78
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.80	0.80	0.76	0.79	0.76	0.76	0.79	0.76	0.82	0.75	0.82	0.75
SILS 1: Wind, North	Leg 2	0.75	0.84	0.80	0.88	0.85	0.87	0.79	0.74	0.79	0.68	0.75	0.63
SILS 1: Wind, West	Leg 2	0.67	0.67	0.64	0.64	0.61	0.61	0.60	0.59	0.64	0.60	0.62	0.60
SILS 1: Wind, South	Leg 2	0.64	0.69	0.65	0.72	0.69	0.66	0.71	0.66	0.71	0.60	0.68	0.59
SILS 1: Wind, East	Leg 2	0.64	0.64	0.59	0.59	0.57	0.57	0.57	0.56	0.61	0.57	0.61	0.58
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.73	0.79	0.75	0.81	0.78	0.79	0.74	0.70	0.74	0.66	0.70	0.64
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.64	0.68	0.64	0.70	0.67	0.64	0.66	0.63	0.67	0.60	0.66	0.60
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.70	0.75	0.71	0.77	0.75	0.76	0.70	0.66	0.72	0.63	0.70	0.61
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.64	0.64	0.61	0.66	0.63	0.61	0.65	0.62	0.66	0.58	0.65	0.59
ULS 5: Dead & Live	Leg 2	0.84	0.84	0.78	0.77	0.73	0.73	0.73	0.72	0.79	0.76	0.81	0.78
Max of All Load Combinations (either leg)		**0.98**	**1.00**	**0.95**	**0.99**	**0.95**	**0.97**	**0.99**	**1.00**	**0.99**	**0.98**	**1.00**	**0.99**

Sicilia Tower (4/4)

		37	38	39	40	41	42	43	44	45	46	47	48	49	50
Analysis Section		37	38	39	40	41	42	43	44	45	46	47	48	49	50
Tower Segment		17	17	18	18	19	19	20	20	21	21	21	22	22	22
Elevation		303	323	323	341	341	357.4	357.4	369.9	369.9	374.447	374.447	383.619	383.619	395.619
Load Combin. Description	**Leg**	**Governing Utilization Ratio**													
ULS 7: Seismic, Longit.	Leg 1	0.97	0.95	0.99	0.95	1.00	0.96	0.92	0.92	0.88	0.89	0.88	0.99	0.99	0.83
ULS 7: Seismic, Trans.	Leg 1	0.92	0.91	0.93	0.92	0.94	0.94	0.90	0.91	0.88	0.88	0.86	0.95	0.95	0.82
SILS 2: Seismic, Longit.	Leg 1	0.90	0.85	0.89	0.82	0.87	0.81	0.77	0.76	0.73	0.73	0.72	0.86	0.86	0.69
SILS 2: Seismic, Trans.	Leg 1	0.82	0.78	0.82	0.76	0.81	0.78	0.75	0.75	0.72	0.73	0.69	0.82	0.82	0.67
ULS 2 & 6 Wind, North	Leg 1	0.74	0.73	0.73	0.77	0.71	0.78	0.74	0.78	0.74	0.76	0.86	0.95	0.95	0.77
ULS 2 & 6 Wind, West	Leg 1	0.76	0.75	0.75	0.78	0.73	0.78	0.74	0.77	0.74	0.75	0.79	0.88	0.88	0.76
ULS 2 & 6 Wind, South	Leg 1	0.79	0.86	0.82	0.95	0.87	0.98	0.93	1.00	0.96	0.98	0.79	0.87	0.87	0.77
ULS 2 & 6 Wind, East	Leg 1	0.76	0.74	0.74	0.78	0.72	0.78	0.74	0.77	0.73	0.75	0.79	0.88	0.88	0.75
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.76	0.74	0.73	0.73	0.71	0.73	0.69	0.72	0.69	0.70	0.84	0.92	0.92	0.76
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.79	0.82	0.80	0.89	0.82	0.91	0.86	0.92	0.88	0.89	0.80	0.88	0.88	0.77
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.75	0.73	0.73	0.73	0.70	0.73	0.69	0.72	0.69	0.70	0.83	0.92	0.92	0.76
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.79	0.82	0.79	0.89	0.82	0.91	0.86	0.91	0.87	0.89	0.79	0.88	0.88	0.77
SILS 1: Wind, North	Leg 1	0.59	0.58	0.58	0.64	0.59	0.66	0.62	0.66	0.63	0.65	0.69	0.78	0.78	0.61
SILS 1: Wind, West	Leg 1	0.61	0.60	0.60	0.61	0.58	0.62	0.59	0.61	0.58	0.59	0.62	0.71	0.71	0.60
SILS 1: Wind, South	Leg 1	0.64	0.71	0.68	0.81	0.74	0.84	0.80	0.87	0.83	0.85	0.61	0.69	0.69	0.62
SILS 1: Wind, East	Leg 1	0.59	0.58	0.58	0.61	0.57	0.61	0.58	0.61	0.58	0.59	0.61	0.70	0.70	0.60
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.61	0.59	0.58	0.59	0.56	0.61	0.57	0.60	0.57	0.58	0.66	0.76	0.76	0.61
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.65	0.67	0.66	0.75	0.69	0.77	0.73	0.78	0.75	0.76	0.62	0.71	0.71	0.62
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.59	0.58	0.58	0.59	0.55	0.60	0.57	0.60	0.57	0.58	0.66	0.75	0.75	0.61
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.62	0.67	0.65	0.74	0.68	0.77	0.72	0.78	0.74	0.76	0.61	0.70	0.70	0.62
ULS 5: Dead & Live	Leg 1	0.79	0.78	0.77	0.80	0.75	0.80	0.75	0.79	0.75	0.76	0.83	0.93	0.93	0.80
ULS 7: Seismic, Longit.	Leg 2	1.00	0.96	1.00	0.93	0.98	0.94	0.89	0.90	0.86	0.87	0.91	1.00	1.00	0.83
ULS 7: Seismic, Trans.	Leg 2	0.90	0.88	0.89	0.90	0.91	0.94	0.90	0.92	0.89	0.90	0.88	0.97	0.97	0.82
SILS 2: Seismic, Longit.	Leg 2	0.92	0.87	0.91	0.82	0.87	0.78	0.75	0.74	0.71	0.71	0.74	0.87	0.87	0.69
SILS 2: Seismic, Trans.	Leg 2	0.82	0.76	0.80	0.74	0.78	0.78	0.74	0.76	0.73	0.74	0.71	0.83	0.83	0.67
ULS 2 & 6 Wind, North	Leg 2	0.78	0.85	0.82	0.95	0.87	0.98	0.93	1.00	0.95	0.98	0.78	0.87	0.87	0.76
ULS 2 & 6 Wind, West	Leg 2	0.76	0.75	0.75	0.78	0.73	0.79	0.74	0.78	0.74	0.75	0.80	0.88	0.88	0.76
ULS 2 & 6 Wind, South	Leg 2	0.75	0.74	0.73	0.76	0.71	0.76	0.72	0.75	0.72	0.73	0.87	0.95	0.95	0.77
ULS 2 & 6 Wind, East	Leg 2	0.76	0.75	0.74	0.78	0.72	0.78	0.74	0.77	0.74	0.75	0.79	0.88	0.88	0.75
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.79	0.82	0.80	0.90	0.83	0.92	0.86	0.92	0.88	0.90	0.79	0.88	0.88	0.77
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.76	0.75	0.74	0.76	0.71	0.76	0.72	0.75	0.71	0.72	0.84	0.92	0.92	0.77
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.79	0.82	0.79	0.89	0.82	0.91	0.86	0.92	0.88	0.90	0.79	0.88	0.88	0.77
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.76	0.74	0.73	0.76	0.71	0.76	0.72	0.75	0.71	0.72	0.84	0.92	0.92	0.76
SILS 1: Wind, North	Leg 2	0.64	0.70	0.67	0.80	0.74	0.84	0.79	0.86	0.82	0.85	0.60	0.69	0.69	0.61
SILS 1: Wind, West	Leg 2	0.61	0.60	0.60	0.62	0.58	0.62	0.59	0.61	0.59	0.59	0.62	0.71	0.71	0.60
SILS 1: Wind, South	Leg 2	0.60	0.58	0.58	0.60	0.56	0.60	0.57	0.60	0.57	0.58	0.70	0.78	0.78	0.62
SILS 1: Wind, East	Leg 2	0.59	0.58	0.58	0.61	0.57	0.62	0.58	0.61	0.58	0.59	0.61	0.70	0.70	0.60
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.64	0.67	0.65	0.75	0.69	0.77	0.72	0.77	0.74	0.76	0.61	0.71	0.71	0.62
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.61	0.59	0.59	0.59	0.57	0.59	0.55	0.58	0.55	0.56	0.67	0.76	0.76	0.62
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.62	0.67	0.64	0.74	0.68	0.76	0.72	0.77	0.74	0.76	0.61	0.70	0.70	0.62
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.59	0.58	0.58	0.59	0.56	0.59	0.55	0.58	0.55	0.56	0.67	0.75	0.75	0.62
ULS 5: Dead & Live	Leg 2	0.79	0.78	0.77	0.80	0.75	0.80	0.76	0.79	0.75	0.76	0.83	0.93	0.93	0.80
Max of All Load Combinations (either leg)		**1.00**	**0.96**	**1.00**	**0.95**	**1.00**	**0.98**	**0.93**	**1.00**	**0.96**	**0.98**	**0.91**	**1.00**	**1.00**	**0.83**

The following plot shows the governing utilization ratios for all load combinations for the Sicilia tower. The values shown are the same as those provided in the tables above.


Governing Sicilia Tower Utilization Ratios for All Load Combinations
450
400
350
300
250
200
150
100
50
0
Elevvation, m
0.50
0.55
0.60
0.65
0.70
0.75
0.80
0.85
0.90
0.95
1.00
1.05
1.10
Utilization Ratio
max of all load combinations

Calabria Tower (1/4)

Load Combin. Description	Leg	1	2	3	4	5	6	7	8	9	10	11	12
Analysis Section		1	2	3	4	5	6	7	8	9	10	11	12
Tower Segment		1	1	2	2	3	3	4	4	5	5	6	6
Elevation		18	28	28	40	40	55	55	71	71	87	87	105
		Governing Utilization Ratio											
ULS 7: Seismic, Longit.	Leg 1	0.99	0.93	1.00	0.92	1.00	0.92	0.99	0.95	0.98	0.94	1.00	0.96
ULS 7: Seismic, Trans.	Leg 1	0.93	0.88	0.94	0.87	0.95	0.89	0.96	0.93	0.96	0.93	0.98	0.95
SILS 2: Seismic, Longit.	Leg 1	0.95	0.88	0.95	0.87	0.95	0.86	0.92	0.83	0.90	0.85	0.93	0.89
SILS 2: Seismic, Trans.	Leg 1	0.87	0.82	0.88	0.82	0.89	0.81	0.86	0.81	0.87	0.82	0.90	0.86
ULS 2 & 6 Wind, North	Leg 1	0.73	0.68	0.76	0.70	0.76	0.75	0.81	0.80	0.82	0.81	0.84	0.84
ULS 2 & 6 Wind, West	Leg 1	0.68	0.66	0.75	0.74	0.80	0.78	0.85	0.83	0.85	0.83	0.86	0.86
ULS 2 & 6 Wind, South	Leg 1	0.92	0.87	0.97	0.91	0.99	0.91	0.98	0.94	0.98	0.94	0.98	1.00
ULS 2 & 6 Wind, East	Leg 1	0.65	0.64	0.72	0.71	0.77	0.76	0.82	0.81	0.83	0.82	0.85	0.86
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.70	0.66	0.74	0.70	0.76	0.75	0.81	0.80	0.82	0.81	0.84	0.83
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.85	0.81	0.90	0.85	0.93	0.87	0.94	0.91	0.94	0.91	0.94	0.96
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.69	0.66	0.73	0.71	0.77	0.75	0.82	0.80	0.82	0.81	0.84	0.83
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.82	0.79	0.88	0.83	0.90	0.84	0.91	0.89	0.92	0.90	0.93	0.95
SILS 1: Wind, North	Leg 1	0.67	0.62	0.68	0.61	0.67	0.63	0.68	0.67	0.68	0.67	0.72	0.70
SILS 1: Wind, West	Leg 1	0.62	0.60	0.64	0.62	0.68	0.66	0.71	0.70	0.72	0.70	0.75	0.73
SILS 1: Wind, South	Leg 1	0.86	0.81	0.90	0.84	0.91	0.82	0.89	0.82	0.87	0.82	0.86	0.88
SILS 1: Wind, East	Leg 1	0.55	0.54	0.59	0.59	0.64	0.64	0.69	0.68	0.70	0.69	0.72	0.72
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.63	0.59	0.66	0.60	0.65	0.63	0.68	0.67	0.69	0.67	0.72	0.70
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.78	0.74	0.82	0.77	0.84	0.77	0.83	0.79	0.82	0.78	0.83	0.83
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.59	0.55	0.61	0.59	0.63	0.63	0.68	0.67	0.68	0.67	0.72	0.70
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.73	0.70	0.78	0.74	0.80	0.74	0.80	0.76	0.80	0.77	0.80	0.82
ULS 5: Dead & Live	Leg 1	0.66	0.65	0.74	0.73	0.79	0.79	0.85	0.84	0.86	0.85	0.88	0.88
ULS 7: Seismic, Longit.	Leg 2	0.990	0.93	0.99	0.91	0.99	0.92	0.99	0.96	0.99	0.93	0.99	0.96
ULS 7: Seismic, Trans.	Leg 2	0.93	0.87	0.93	0.86	0.93	0.88	0.96	0.93	0.96	0.91	0.96	0.96
SILS 2: Seismic, Longit.	Leg 2	0.94	0.89	0.95	0.87	0.95	0.86	0.92	0.83	0.90	0.83	0.92	0.89
SILS 2: Seismic, Trans.	Leg 2	0.87	0.82	0.88	0.82	0.89	0.81	0.87	0.80	0.86	0.81	0.89	0.85
ULS 2 & 6 Wind, North	Leg 2	0.87	0.84	0.93	0.89	0.97	0.90	0.98	0.92	0.97	0.94	0.98	0.98
ULS 2 & 6 Wind, West	Leg 2	0.67	0.64	0.73	0.72	0.78	0.77	0.83	0.81	0.84	0.83	0.87	0.86
ULS 2 & 6 Wind, South	Leg 2	0.75	0.72	0.80	0.74	0.81	0.74	0.80	0.79	0.81	0.81	0.85	0.85
ULS 2 & 6 Wind, East	Leg 2	0.65	0.64	0.72	0.71	0.77	0.76	0.82	0.81	0.83	0.82	0.85	0.85
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.81	0.78	0.87	0.83	0.90	0.87	0.93	0.90	0.93	0.91	0.95	0.94
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.73	0.70	0.78	0.73	0.79	0.74	0.80	0.79	0.81	0.81	0.85	0.84
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.80	0.77	0.86	0.82	0.89	0.85	0.91	0.88	0.91	0.90	0.93	0.94
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.69	0.66	0.74	0.70	0.76	0.74	0.80	0.79	0.81	0.81	0.85	0.84
SILS 1: Wind, North	Leg 2	0.81	0.77	0.86	0.81	0.88	0.82	0.89	0.82	0.87	0.82	0.86	0.86
SILS 1: Wind, West	Leg 2	0.61	0.59	0.64	0.61	0.67	0.65	0.70	0.69	0.72	0.70	0.75	0.72
SILS 1: Wind, South	Leg 2	0.68	0.63	0.70	0.65	0.70	0.63	0.68	0.67	0.68	0.67	0.72	0.71
SILS 1: Wind, East	Leg 2	0.55	0.54	0.59	0.58	0.63	0.62	0.67	0.67	0.68	0.69	0.72	0.72
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.74	0.71	0.79	0.75	0.81	0.77	0.82	0.78	0.82	0.78	0.83	0.82
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.65	0.61	0.68	0.63	0.69	0.63	0.68	0.66	0.70	0.67	0.72	0.71
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.69	0.67	0.74	0.71	0.77	0.73	0.79	0.76	0.80	0.77	0.81	0.81
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.60	0.58	0.64	0.60	0.65	0.63	0.68	0.67	0.68	0.67	0.72	0.71
ULS 5: Dead & Live	Leg 2	0.66	0.65	0.74	0.73	0.79	0.79	0.85	0.84	0.86	0.85	0.88	0.87
Max of All Load Combinations (either leg)		**0.99**	**0.93**	**1.00**	**0.92**	**1.00**	**0.92**	**0.99**	**0.96**	**0.99**	**0.94**	**1.00**	**1.00**

Calabria Tower (2/4)

	Analysis Section	13	14	15	16	17	18	19	20	21	22	23	24
	Tower Segment	7	7	8	8	8	8	9	9	10	10	11	11
	Elevation	105	123.65	123.65	124	124	143	143	163	163	183	183	203
Load Combin. Description	**Leg**	**Governing Utilization Ratio**											
ULS 7: Seismic, Longit.	Leg 1	0.98	0.95	0.99	0.99	0.99	0.98	0.98	0.98	0.98	1.00	0.98	1.00
ULS 7: Seismic, Trans.	Leg 1	0.95	0.92	0.97	0.97	0.97	0.95	0.97	0.96	0.97	0.96	0.96	0.96
SILS 2: Seismic, Longit.	Leg 1	0.89	0.87	0.90	0.90	0.90	0.89	0.86	0.88	0.89	0.92	0.90	0.92
SILS 2: Seismic, Trans.	Leg 1	0.86	0.84	0.88	0.88	0.88	0.87	0.84	0.83	0.84	0.87	0.86	0.87
ULS 2 & 6 Wind, North	Leg 1	0.78	0.77	0.83	0.82	0.82	0.75	0.81	0.79	0.82	0.81	0.81	0.84
ULS 2 & 6 Wind, West	Leg 1	0.80	0.79	0.78	0.78	0.78	0.76	0.82	0.80	0.83	0.82	0.82	0.81
ULS 2 & 6 Wind, South	Leg 1	0.91	0.97	0.99	0.99	0.99	0.91	0.99	0.93	0.97	0.91	0.91	0.97
ULS 2 & 6 Wind, East	Leg 1	0.79	0.80	0.79	0.79	0.79	0.78	0.84	0.82	0.84	0.84	0.84	0.83
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.77	0.77	0.78	0.78	0.78	0.75	0.81	0.79	0.82	0.81	0.81	0.83
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.88	0.91	0.90	0.90	0.90	0.86	0.93	0.88	0.92	0.87	0.87	0.92
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.77	0.77	0.78	0.78	0.78	0.75	0.81	0.79	0.82	0.82	0.82	0.84
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.87	0.91	0.91	0.91	0.91	0.87	0.94	0.90	0.94	0.90	0.90	0.93
SILS 1: Wind, North	Leg 1	0.68	0.66	0.73	0.73	0.73	0.66	0.69	0.66	0.67	0.67	0.66	0.67
SILS 1: Wind, West	Leg 1	0.71	0.68	0.70	0.70	0.70	0.67	0.70	0.67	0.68	0.67	0.67	0.66
SILS 1: Wind, South	Leg 1	0.81	0.89	0.91	0.91	0.91	0.84	0.90	0.81	0.85	0.77	0.77	0.82
SILS 1: Wind, East	Leg 1	0.69	0.69	0.71	0.71	0.71	0.69	0.71	0.69	0.70	0.69	0.69	0.69
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.68	0.66	0.68	0.68	0.68	0.66	0.68	0.66	0.67	0.67	0.66	0.66
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.78	0.81	0.82	0.82	0.82	0.77	0.81	0.75	0.79	0.73	0.73	0.77
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.68	0.66	0.68	0.68	0.68	0.66	0.68	0.67	0.67	0.67	0.67	0.67
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.77	0.82	0.83	0.83	0.83	0.80	0.83	0.78	0.81	0.76	0.76	0.77
ULS 5: Dead & Live	Leg 1	0.82	0.82	0.80	0.80	0.80	0.78	0.84	0.82	0.85	0.85	0.85	0.86
ULS 7: Seismic, Longit.	Leg 2	0.97	0.94	0.97	0.97	0.97	0.96	0.95	0.97	0.97	0.99	0.97	0.98
ULS 7: Seismic, Trans.	Leg 2	0.94	0.94	0.95	0.94	0.94	0.93	0.94	0.94	0.95	0.95	0.95	0.95
SILS 2: Seismic, Longit.	Leg 2	0.89	0.85	0.89	0.89	0.89	0.87	0.84	0.89	0.89	0.92	0.90	0.91
SILS 2: Seismic, Trans.	Leg 2	0.85	0.84	0.86	0.86	0.86	0.84	0.82	0.84	0.85	0.87	0.86	0.86
ULS 2 & 6 Wind, North	Leg 2	0.91	0.98	0.96	0.96	0.96	0.90	0.97	0.92	0.95	0.91	0.91	0.97
ULS 2 & 6 Wind, West	Leg 2	0.80	0.81	0.77	0.77	0.77	0.76	0.82	0.79	0.81	0.81	0.82	0.81
ULS 2 & 6 Wind, South	Leg 2	0.79	0.80	0.87	0.87	0.87	0.77	0.84	0.79	0.82	0.82	0.82	0.83
ULS 2 & 6 Wind, East	Leg 2	0.79	0.82	0.78	0.78	0.78	0.77	0.83	0.81	0.84	0.84	0.84	0.84
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.88	0.91	0.88	0.88	0.88	0.85	0.91	0.87	0.90	0.87	0.87	0.92
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.78	0.80	0.81	0.81	0.81	0.75	0.80	0.79	0.82	0.82	0.82	0.82
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.87	0.92	0.89	0.89	0.89	0.86	0.93	0.89	0.92	0.89	0.89	0.92
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.78	0.80	0.82	0.82	0.82	0.76	0.82	0.79	0.82	0.82	0.82	0.83
SILS 1: Wind, North	Leg 2	0.81	0.88	0.87	0.87	0.87	0.81	0.86	0.80	0.83	0.77	0.77	0.82
SILS 1: Wind, West	Leg 2	0.71	0.69	0.67	0.67	0.67	0.66	0.68	0.66	0.67	0.67	0.67	0.66
SILS 1: Wind, South	Leg 2	0.69	0.68	0.78	0.77	0.77	0.68	0.74	0.67	0.68	0.67	0.67	0.67
SILS 1: Wind, East	Leg 2	0.69	0.69	0.68	0.68	0.68	0.68	0.69	0.69	0.70	0.69	0.69	0.69
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.77	0.81	0.78	0.78	0.78	0.73	0.79	0.75	0.77	0.73	0.73	0.76
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.69	0.67	0.72	0.71	0.71	0.66	0.69	0.67	0.67	0.67	0.66	0.66
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.76	0.82	0.79	0.78	0.78	0.78	0.80	0.77	0.79	0.76	0.75	0.77
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.69	0.67	0.72	0.72	0.72	0.68	0.71	0.67	0.68	0.67	0.67	0.67
ULS 5: Dead & Live	Leg 2	0.82	0.82	0.79	0.79	0.79	0.78	0.83	0.82	0.84	0.85	0.85	0.86
Max of All Load Combinations (either leg)		**0.98**	**0.98**	**0.99**	**0.99**	**0.99**	**0.98**	**0.99**	**0.98**	**0.98**	**1.00**	**0.98**	**1.00**

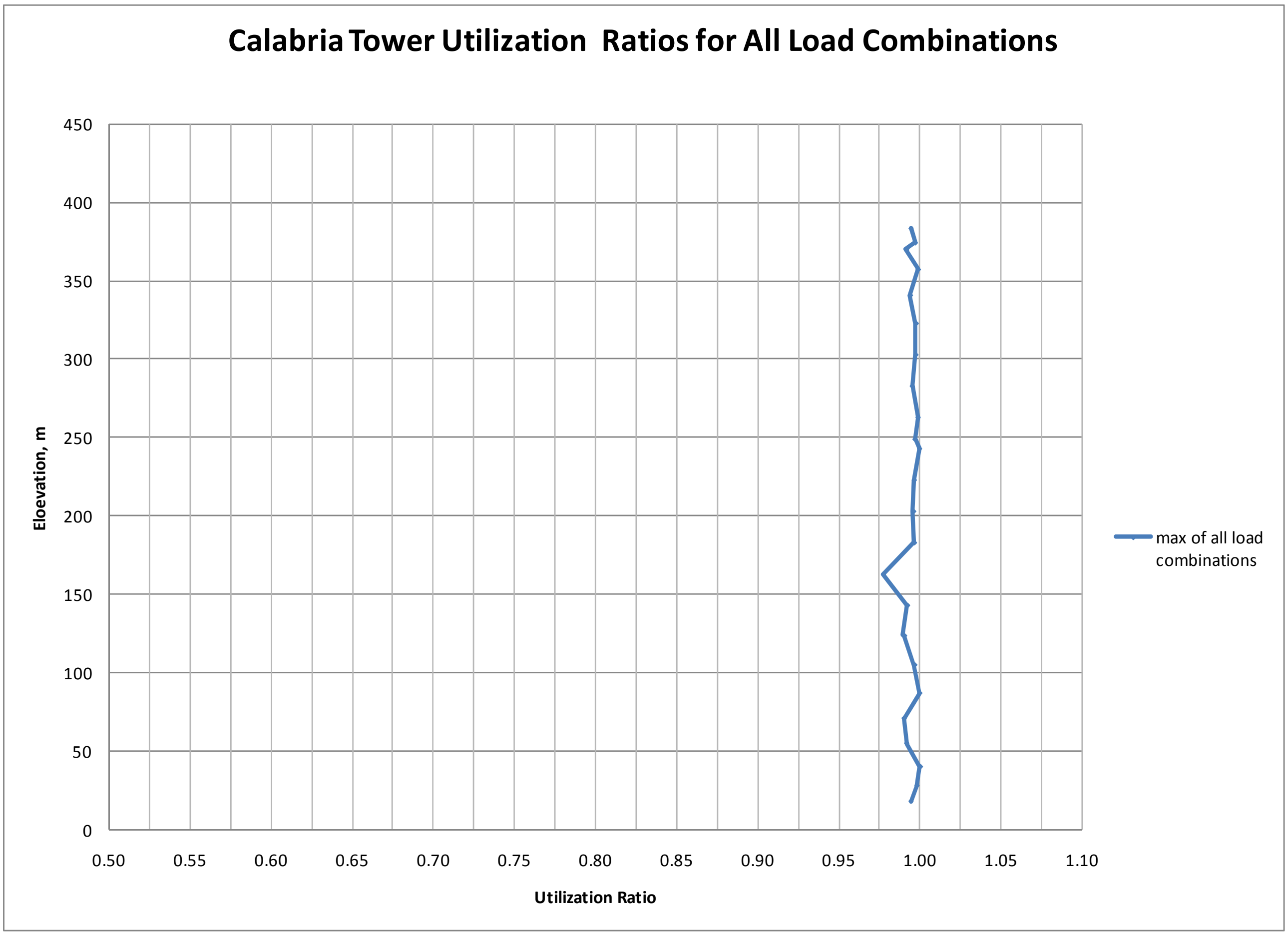
Calabria Tower (3/4)

Analysis Section		25	26	27	28	29	30	31	32	33	34	35	36
Tower Segment		12	12	13	13	14	14	14	14	15	15	16	16
Elevation		203	223	223	243	243	249.048	249.048	263	263	283	283	303
Load Combin. Description	**Leg**	**Governing Utilization Ratio**											
ULS 7: Seismic, Longit.	Leg 1	0.98	1.00	0.97	1.00	0.97	0.98	1.00	1.00	1.00	0.98	0.99	0.97
ULS 7: Seismic, Trans.	Leg 1	0.95	0.95	0.92	0.95	0.92	0.92	0.95	0.94	0.96	0.94	0.96	0.93
SILS 2: Seismic, Longit.	Leg 1	0.91	0.93	0.91	0.93	0.90	0.90	0.90	0.91	0.90	0.90	0.90	0.88
SILS 2: Seismic, Trans.	Leg 1	0.86	0.88	0.86	0.86	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.82
ULS 2 & 6 Wind, North	Leg 1	0.80	0.84	0.79	0.85	0.82	0.84	0.76	0.74	0.80	0.76	0.82	0.77
ULS 2 & 6 Wind, West	Leg 1	0.80	0.79	0.73	0.73	0.71	0.71	0.72	0.71	0.78	0.76	0.78	0.77
ULS 2 & 6 Wind, South	Leg 1	0.90	0.99	0.91	0.99	0.97	0.99	0.92	0.87	0.95	0.83	0.91	0.81
ULS 2 & 6 Wind, East	Leg 1	0.82	0.82	0.76	0.76	0.74	0.74	0.75	0.73	0.81	0.79	0.80	0.79
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.80	0.82	0.75	0.80	0.78	0.79	0.72	0.71	0.78	0.76	0.79	0.77
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.85	0.91	0.84	0.90	0.88	0.89	0.85	0.82	0.89	0.80	0.86	0.80
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.81	0.84	0.78	0.83	0.80	0.81	0.72	0.72	0.79	0.78	0.80	0.78
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.88	0.94	0.87	0.93	0.89	0.91	0.88	0.84	0.91	0.83	0.86	0.82
SILS 1: Wind, North	Leg 1	0.65	0.71	0.66	0.73	0.71	0.72	0.65	0.61	0.66	0.61	0.66	0.61
SILS 1: Wind, West	Leg 1	0.65	0.64	0.61	0.61	0.59	0.58	0.58	0.57	0.62	0.61	0.62	0.61
SILS 1: Wind, South	Leg 1	0.77	0.87	0.81	0.89	0.86	0.89	0.81	0.75	0.82	0.70	0.75	0.66
SILS 1: Wind, East	Leg 1	0.68	0.67	0.66	0.65	0.63	0.62	0.62	0.61	0.65	0.63	0.65	0.63
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.65	0.67	0.62	0.67	0.65	0.66	0.58	0.57	0.62	0.61	0.63	0.61
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.71	0.78	0.72	0.78	0.76	0.77	0.72	0.68	0.74	0.64	0.70	0.63
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.66	0.70	0.66	0.71	0.69	0.70	0.62	0.61	0.64	0.63	0.64	0.62
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.75	0.81	0.76	0.82	0.79	0.81	0.76	0.72	0.77	0.68	0.71	0.67
ULS 5: Dead & Live	Leg 1	0.83	0.83	0.76	0.76	0.74	0.73	0.74	0.73	0.82	0.80	0.82	0.81
ULS 7: Seismic, Longit.	Leg 2	0.97	0.99	0.97	0.99	0.96	0.97	0.97	0.97	0.97	0.97	0.98	0.96
ULS 7: Seismic, Trans.	Leg 2	0.94	0.94	0.92	0.94	0.92	0.92	0.92	0.92	0.93	0.92	0.93	0.92
SILS 2: Seismic, Longit.	Leg 2	0.90	0.92	0.90	0.91	0.88	0.89	0.89	0.90	0.90	0.89	0.90	0.87
SILS 2: Seismic, Trans.	Leg 2	0.85	0.86	0.85	0.85	0.83	0.83	0.83	0.83	0.83	0.82	0.83	0.81
ULS 2 & 6 Wind, North	Leg 2	0.90	0.98	0.91	0.99	0.96	0.98	0.91	0.86	0.94	0.83	0.90	0.81
ULS 2 & 6 Wind, West	Leg 2	0.80	0.80	0.73	0.73	0.71	0.71	0.71	0.70	0.78	0.76	0.78	0.77
ULS 2 & 6 Wind, South	Leg 2	0.80	0.82	0.76	0.79	0.77	0.78	0.84	0.80	0.87	0.77	0.84	0.77
ULS 2 & 6 Wind, East	Leg 2	0.82	0.83	0.77	0.77	0.75	0.75	0.74	0.73	0.80	0.78	0.80	0.79
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.86	0.91	0.83	0.90	0.87	0.89	0.84	0.80	0.88	0.80	0.85	0.80
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.80	0.80	0.74	0.77	0.75	0.75	0.77	0.75	0.82	0.77	0.81	0.77
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.89	0.94	0.87	0.93	0.89	0.91	0.86	0.82	0.90	0.82	0.85	0.82
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.81	0.82	0.76	0.76	0.74	0.74	0.80	0.77	0.84	0.78	0.81	0.79
SILS 1: Wind, North	Leg 2	0.77	0.86	0.81	0.89	0.86	0.88	0.79	0.74	0.80	0.69	0.74	0.65
SILS 1: Wind, West	Leg 2	0.65	0.65	0.61	0.61	0.59	0.58	0.58	0.57	0.62	0.61	0.62	0.61
SILS 1: Wind, South	Leg 2	0.65	0.70	0.65	0.67	0.65	0.67	0.73	0.68	0.73	0.62	0.67	0.61
SILS 1: Wind, East	Leg 2	0.68	0.68	0.66	0.65	0.63	0.63	0.62	0.61	0.65	0.63	0.64	0.63
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.72	0.78	0.72	0.78	0.75	0.77	0.71	0.67	0.73	0.64	0.69	0.63
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.65	0.66	0.62	0.62	0.60	0.61	0.65	0.61	0.67	0.61	0.65	0.61
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.75	0.81	0.76	0.82	0.79	0.81	0.74	0.70	0.76	0.67	0.70	0.66
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.67	0.69	0.65	0.65	0.63	0.64	0.68	0.65	0.70	0.63	0.65	0.63
ULS 5: Dead & Live	Leg 2	0.83	0.83	0.76	0.76	0.74	0.74	0.74	0.73	0.81	0.80	0.82	0.81
Max of All Load Combinations (either leg)		**0.98**	**1.00**	**0.97**	**1.00**	**0.97**	**0.99**	**1.00**	**1.00**	**1.00**	**0.98**	**0.99**	**0.97**

Calabria Tower (4/4)

Load Combin. Description	Leg	37	38	39	40	41	42	43	44	45	46	47	48
Analysis Section		37	38	39	40	41	42	43	44	45	46	47	48
Tower Segment		17	17	18	18	19	19	20	20	21	21	21	22
Elevation		303	323	323	341	341	357.4	357.4	369.9	369.9	374.447	374.447	383.619
Load Combin. Description	**Leg**	**Governing Utilization Ratio**											
ULS 7: Seismic, Longit.	Leg 1	1.00	0.97	1.00	0.93	0.99	0.94	0.97	0.88	0.85	0.86	0.88	0.98
ULS 7: Seismic, Trans.	Leg 1	0.94	0.91	0.94	0.91	0.94	0.93	0.92	0.89	0.88	0.90	0.88	0.98
SILS 2: Seismic, Longit.	Leg 1	0.91	0.86	0.89	0.82	0.86	0.78	0.83	0.74	0.69	0.71	0.70	0.84
SILS 2: Seismic, Trans.	Leg 1	0.85	0.80	0.83	0.76	0.80	0.76	0.78	0.74	0.72	0.74	0.69	0.81
ULS 2 & 6 Wind, North	Leg 1	0.76	0.75	0.75	0.78	0.77	0.80	0.74	0.78	0.76	0.77	0.87	0.97
ULS 2 & 6 Wind, West	Leg 1	0.77	0.77	0.76	0.79	0.78	0.80	0.73	0.77	0.74	0.76	0.80	0.90
ULS 2 & 6 Wind, South	Leg 1	0.81	0.88	0.84	0.96	0.90	1.00	0.92	0.99	0.97	1.00	0.80	0.89
ULS 2 & 6 Wind, East	Leg 1	0.78	0.77	0.78	0.80	0.79	0.81	0.74	0.77	0.75	0.76	0.81	0.90
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 1	0.76	0.75	0.75	0.76	0.76	0.77	0.69	0.72	0.70	0.71	0.85	0.94
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 1	0.79	0.85	0.82	0.90	0.85	0.92	0.85	0.91	0.89	0.91	0.80	0.89
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 1	0.78	0.76	0.76	0.77	0.77	0.78	0.70	0.72	0.70	0.71	0.85	0.94
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 1	0.81	0.85	0.83	0.90	0.86	0.93	0.86	0.91	0.89	0.91	0.81	0.90
SILS 1: Wind, North	Leg 1	0.60	0.59	0.59	0.65	0.61	0.67	0.62	0.66	0.65	0.66	0.70	0.80
SILS 1: Wind, West	Leg 1	0.60	0.60	0.59	0.62	0.61	0.63	0.58	0.60	0.59	0.60	0.62	0.72
SILS 1: Wind, South	Leg 1	0.65	0.73	0.70	0.81	0.76	0.86	0.79	0.86	0.85	0.87	0.62	0.71
SILS 1: Wind, East	Leg 1	0.63	0.61	0.62	0.64	0.63	0.65	0.59	0.61	0.59	0.60	0.63	0.73
SILS 1: Wind, 45 deg (+,+)	Leg 1	0.60	0.59	0.59	0.60	0.60	0.62	0.57	0.60	0.58	0.59	0.67	0.76
SILS 1: Wind, 45 deg (+,-)	Leg 1	0.63	0.69	0.66	0.75	0.70	0.78	0.72	0.77	0.76	0.77	0.62	0.72
SILS 1: Wind, 45 deg (-,+)	Leg 1	0.62	0.60	0.60	0.61	0.61	0.63	0.58	0.60	0.59	0.60	0.67	0.78
SILS 1: Wind, 45 deg (-,-)	Leg 1	0.66	0.70	0.69	0.75	0.72	0.78	0.73	0.77	0.76	0.78	0.63	0.73
ULS 5: Dead & Live	Leg 1	0.81	0.79	0.79	0.82	0.81	0.83	0.75	0.78	0.76	0.77	0.84	0.94
ULS 7: Seismic, Longit.	Leg 2	1.00	0.96	0.99	0.93	0.99	0.95	0.97	0.92	0.87	0.89	0.90	0.99
ULS 7: Seismic, Trans.	Leg 2	0.93	0.90	0.93	0.92	0.94	0.95	0.92	0.92	0.89	0.91	0.89	0.98
SILS 2: Seismic, Longit.	Leg 2	0.90	0.85	0.88	0.81	0.85	0.78	0.82	0.77	0.71	0.72	0.72	0.85
SILS 2: Seismic, Trans.	Leg 2	0.83	0.79	0.82	0.75	0.79	0.78	0.78	0.75	0.73	0.75	0.71	0.83
ULS 2 & 6 Wind, North	Leg 2	0.80	0.87	0.84	0.96	0.90	1.00	0.92	0.99	0.97	1.00	0.79	0.89
ULS 2 & 6 Wind, West	Leg 2	0.77	0.77	0.76	0.79	0.77	0.80	0.74	0.77	0.75	0.76	0.80	0.90
ULS 2 & 6 Wind, South	Leg 2	0.77	0.76	0.75	0.78	0.76	0.78	0.71	0.74	0.72	0.73	0.88	0.97
ULS 2 & 6 Wind, East	Leg 2	0.78	0.77	0.78	0.80	0.79	0.81	0.74	0.77	0.75	0.76	0.81	0.90
ULS 2 & 6 Wind, 45 deg (+,+)	Leg 2	0.79	0.84	0.82	0.90	0.85	0.93	0.86	0.91	0.89	0.91	0.80	0.89
ULS 2 & 6 Wind, 45 deg (+,-)	Leg 2	0.77	0.75	0.75	0.77	0.76	0.78	0.71	0.74	0.72	0.73	0.85	0.94
ULS 2 & 6 Wind, 45 deg (-,+)	Leg 2	0.81	0.85	0.83	0.91	0.87	0.93	0.86	0.92	0.90	0.92	0.80	0.90
ULS 2 & 6 Wind, 45 deg (-,-)	Leg 2	0.78	0.76	0.76	0.77	0.77	0.78	0.71	0.74	0.72	0.73	0.86	0.94
SILS 1: Wind, North	Leg 2	0.65	0.72	0.69	0.81	0.75	0.85	0.79	0.85	0.84	0.86	0.61	0.71
SILS 1: Wind, West	Leg 2	0.60	0.60	0.59	0.62	0.60	0.63	0.58	0.61	0.59	0.60	0.62	0.72
SILS 1: Wind, South	Leg 2	0.61	0.59	0.59	0.61	0.60	0.62	0.57	0.60	0.58	0.60	0.71	0.80
SILS 1: Wind, East	Leg 2	0.63	0.61	0.62	0.64	0.63	0.65	0.59	0.61	0.59	0.60	0.63	0.73
SILS 1: Wind, 45 deg (+,+)	Leg 2	0.62	0.69	0.65	0.75	0.69	0.77	0.72	0.77	0.75	0.77	0.62	0.72
SILS 1: Wind, 45 deg (+,-)	Leg 2	0.61	0.59	0.59	0.61	0.60	0.61	0.55	0.57	0.56	0.56	0.68	0.77
SILS 1: Wind, 45 deg (-,+)	Leg 2	0.66	0.69	0.69	0.75	0.72	0.78	0.72	0.77	0.76	0.77	0.62	0.73
SILS 1: Wind, 45 deg (-,-)	Leg 2	0.62	0.61	0.60	0.61	0.61	0.61	0.57	0.57	0.56	0.57	0.68	0.78
ULS 5: Dead & Live	Leg 2	0.81	0.79	0.79	0.82	0.80	0.82	0.75	0.78	0.76	0.77	0.84	0.94
Max of All Load Combinations (either leg)		**1.00**	**0.97**	**1.00**	**0.96**	**0.99**	**1.00**	**0.97**	**0.99**	**0.97**	**1.00**	**0.90**	**0.99**

The following plot shows the governing utilization ratios for all load combinations for the Calabria tower. The values shown are the same as those provided in the tables above.



Calabria Tower Utilization Ratios for All Load Combinations
Eloevation, m
450
400
350
300
250
200
150
100
50
0
0.50
0.55
0.60
0.65
0.70
0.75
0.80
0.85
0.90
0.95
1.00
1.05
1.10
Utilization Ratio
max of all load combinations

5.1.1.8 Verification for the Envelope of All Time-History Seismic Results

Although the tower legs were designed for the mean results the eight time-history inputs that were analyzed, on account of the exceptional importance of the structure, the tower legs were also verified for the envelope of all time-history analysis results, assuming all partial safety factors are equal to 1.0. Maximum primary force effects from each time-history input are plotted on the Specialist Technical Design Report. Maximum utilization ratios for each time-history in put and directional combination are presented below for each tower leg cross section.

The maximum utilization ratios for each case are plotted after the tabular data.

The maximum calculated utilization ratio is 1.07, and occurs in Calabria tower leg 1 segment 17 and is caused by Sicilia time-history input 3. A detailed finite element model of this segment was created and analysed for the critical load combination to assess the potential extent of damage caused by the apparent overstress. The results of this analysis are described in Section 5.3.8.

Sicilia Tower Leg 1

EL (m)	Time History Input																Maximum
	Longitudinal Combination - 1.0 x Long + 0.8 x Trans + 0.75 x Vert								Transverse Combination - 0.8 x Long + 1.0 x Trans + 0.75 x Vert								
	S1	S2	S3	S4	C1	C2	C3	C4	S1	S2	S3	S4	C1	C2	C3	C4	
18	0.99	0.96	0.96	0.95	0.89	0.86	0.84	0.91	0.90	0.89	0.88	0.87	0.82	0.82	0.79	0.84	0.99
28	0.92	0.87	0.90	0.88	0.83	0.80	0.78	0.85	0.83	0.82	0.83	0.81	0.77	0.76	0.75	0.79	0.92
28	0.98	0.94	0.96	0.94	0.89	0.86	0.84	0.91	0.89	0.88	0.89	0.87	0.83	0.82	0.81	0.85	0.98
40	0.90	0.84	0.89	0.87	0.85	0.82	0.79	0.87	0.83	0.79	0.82	0.81	0.79	0.79	0.76	0.83	0.90
40	0.96	0.90	0.95	0.94	0.91	0.88	0.85	0.93	0.88	0.85	0.88	0.87	0.85	0.85	0.83	0.89	0.96
55	0.86	0.81	0.85	0.87	0.85	0.84	0.79	0.89	0.84	0.78	0.80	0.82	0.80	0.83	0.79	0.87	0.89
55	0.93	0.87	0.91	0.94	0.91	0.91	0.87	0.97	0.92	0.85	0.87	0.88	0.87	0.91	0.86	0.94	0.97
71	0.89	0.86	0.86	0.86	0.84	0.91	0.81	0.92	0.88	0.84	0.83	0.84	0.82	0.90	0.80	0.90	0.92
71	0.93	0.89	0.91	0.91	0.91	0.95	0.85	0.96	0.92	0.88	0.87	0.88	0.87	0.94	0.84	0.94	0.96
87	0.88	0.89	0.88	0.90	0.83	0.93	0.81	0.91	0.86	0.87	0.84	0.87	0.82	0.92	0.81	0.89	0.93
87	0.93	0.92	0.94	0.95	0.88	0.96	0.84	0.94	0.89	0.90	0.88	0.90	0.85	0.95	0.83	0.92	0.96
105	0.92	0.93	0.90	0.93	0.88	0.98	0.85	0.89	0.90	0.90	0.87	0.90	0.85	0.96	0.87	0.88	0.98
105	0.93	0.91	0.91	0.96	0.91	0.95	0.84	0.90	0.88	0.88	0.86	0.90	0.86	0.92	0.83	0.85	0.96
123.65	0.92	0.92	0.87	0.93	0.93	0.97	0.87	0.87	0.87	0.89	0.83	0.88	0.88	0.94	0.88	0.85	0.97
123.65	0.91	0.93	0.89	0.94	0.93	0.95	0.87	0.87	0.85	0.90	0.86	0.89	0.88	0.94	0.89	0.84	0.95
124	0.91	0.93	0.89	0.94	0.93	0.94	0.87	0.86	0.85	0.89	0.86	0.89	0.88	0.94	0.89	0.84	0.94
124	0.91	0.93	0.89	0.94	0.93	0.94	0.87	0.86	0.85	0.89	0.86	0.89	0.88	0.94	0.89	0.84	0.94
143	0.88	0.91	0.91	0.86	0.87	0.87	0.83	0.84	0.83	0.88	0.87	0.83	0.81	0.85	0.84	0.80	0.91
143	0.92	0.96	0.95	0.91	0.91	0.92	0.89	0.88	0.87	0.92	0.91	0.87	0.86	0.91	0.91	0.86	0.96
163	0.97	0.97	0.95	0.91	0.84	0.86	0.88	0.88	0.92	0.91	0.89	0.86	0.81	0.83	0.84	0.84	0.97
163	0.96	0.98	0.96	0.87	0.83	0.89	0.89	0.90	0.92	0.94	0.91	0.84	0.82	0.88	0.87	0.89	0.98
183	0.98	0.95	0.92	0.89	0.90	0.89	0.88	0.91	0.93	0.91	0.88	0.84	0.86	0.88	0.85	0.90	0.98
183	0.99	0.96	0.93	0.90	0.90	0.90	0.89	0.93	0.94	0.92	0.89	0.84	0.87	0.88	0.86	0.91	0.99
203	0.98	0.89	0.93	0.90	0.95	0.91	0.87	0.94	0.93	0.87	0.88	0.86	0.91	0.89	0.84	0.92	0.98
203	0.96	0.88	0.92	0.89	0.93	0.88	0.85	0.92	0.91	0.85	0.87	0.84	0.90	0.87	0.82	0.89	0.96
223	0.94	0.93	0.92	0.99	0.96	0.96	0.87	0.95	0.89	0.89	0.88	0.94	0.92	0.94	0.83	0.90	0.99
223	0.89	0.88	0.88	0.94	0.92	0.91	0.83	0.90	0.85	0.85	0.84	0.89	0.88	0.89	0.80	0.86	0.94
243	0.94	0.97	0.96	**1.05**	0.92	0.98	0.85	0.92	0.89	0.93	0.91	0.98	0.88	0.95	0.83	0.88	**1.05**
243	0.89	0.92	0.90	0.99	0.87	0.93	0.80	0.87	0.84	0.88	0.86	0.93	0.85	0.90	0.80	0.83	0.99
249.048	0.90	0.94	0.93	**1.02**	0.87	0.95	0.82	0.88	0.85	0.90	0.87	0.95	0.83	0.91	0.81	0.84	**1.02**
249.048	0.92	0.92	0.97	**1.00**	0.89	0.89	0.82	0.91	0.87	0.87	0.88	0.92	0.84	0.87	0.80	0.90	**1.00**
263	0.91	0.93	0.98	**1.03**	0.87	0.90	0.84	0.90	0.86	0.88	0.90	0.94	0.82	0.85	0.81	0.87	**1.03**
263	0.91	0.93	0.98	**1.02**	0.87	0.90	0.83	0.92	0.90	0.87	0.90	0.93	0.85	0.86	0.85	0.92	**1.02**
283	0.89	0.93	0.98	**1.03**	0.85	0.89	0.83	0.88	0.84	0.87	0.90	0.94	0.82	0.83	0.80	0.85	**1.03**
283	0.91	0.94	0.99	**1.04**	0.87	0.90	0.84	0.89	0.86	0.88	0.91	0.95	0.84	0.86	0.86	0.89	**1.04**
303	0.87	0.94	0.96	**1.03**	0.84	0.89	0.81	0.85	0.83	0.88	0.88	0.94	0.81	0.84	0.78	0.83	**1.03**
303	0.87	0.95	0.97	**1.04**	0.85	0.90	0.82	0.86	0.84	0.89	0.89	0.95	0.82	0.85	0.78	0.84	**1.04**
323	0.87	0.92	0.92	**1.01**	0.84	0.87	0.76	0.85	0.84	0.86	0.86	0.92	0.81	0.83	0.74	0.83	**1.01**
323	0.90	0.95	0.97	**1.05**	0.88	0.91	0.79	0.87	0.85	0.89	0.90	0.96	0.81	0.85	0.76	0.84	**1.05**
341	0.87	0.89	0.90	0.98	0.89	0.85	0.79	0.86	0.86	0.85	0.85	0.90	0.82	0.83	0.80	0.85	0.98
341	0.91	0.94	0.96	**1.04**	0.94	0.90	0.78	0.88	0.87	0.88	0.90	0.95	0.84	0.85	0.74	0.85	**1.04**
357.4	0.90	0.88	0.87	0.95	0.89	0.85	0.81	0.90	0.90	0.85	0.85	0.89	0.85	0.83	0.83	0.90	0.95
357.4	0.86	0.84	0.84	0.92	0.85	0.81	0.77	0.85	0.86	0.81	0.81	0.85	0.81	0.79	0.79	0.86	0.92
369.9	0.87	0.82	0.80	0.87	0.84	0.81	0.82	0.87	0.87	0.81	0.80	0.83	0.84	0.83	0.84	0.88	0.88
369.9	0.83	0.78	0.76	0.83	0.80	0.77	0.78	0.84	0.84	0.77	0.76	0.80	0.81	0.80	0.81	0.85	0.85
374.447	0.83	0.78	0.76	0.82	0.81	0.78	0.80	0.84	0.84	0.79	0.77	0.80	0.82	0.82	0.82	0.85	0.85
374.447	0.84	0.82	0.80	0.83	0.79	0.80	0.82	0.77	0.82	0.81	0.79	0.82	0.76	0.78	0.81	0.77	0.84
383.619	0.94	0.91	0.92	0.97	0.92	0.89	0.89	0.86	0.90	0.89	0.88	0.91	0.88	0.86	0.89	0.85	0.97
383.619	0.94	0.91	0.92	0.97	0.92	0.89	0.89	0.86	0.90	0.89	0.88	0.91	0.88	0.86	0.89	0.85	0.97
Average	0.91	0.91	0.91	0.94	0.88	0.89	0.83	0.89	0.87	0.87	0.86	0.88	0.84	0.87	0.82	0.86	0.97
Maximum	0.99	0.98	0.99	1.05	0.96	0.98	0.89	0.97	0.94	0.94	0.91	0.98	0.92	0.96	0.91	0.94	1.05
Minimum	0.83	0.78	0.76	0.82	0.79	0.77	0.76	0.77	0.82	0.77	0.76	0.80	0.76	0.76	0.74	0.77	0.84

Sicilia Tower Leg 2

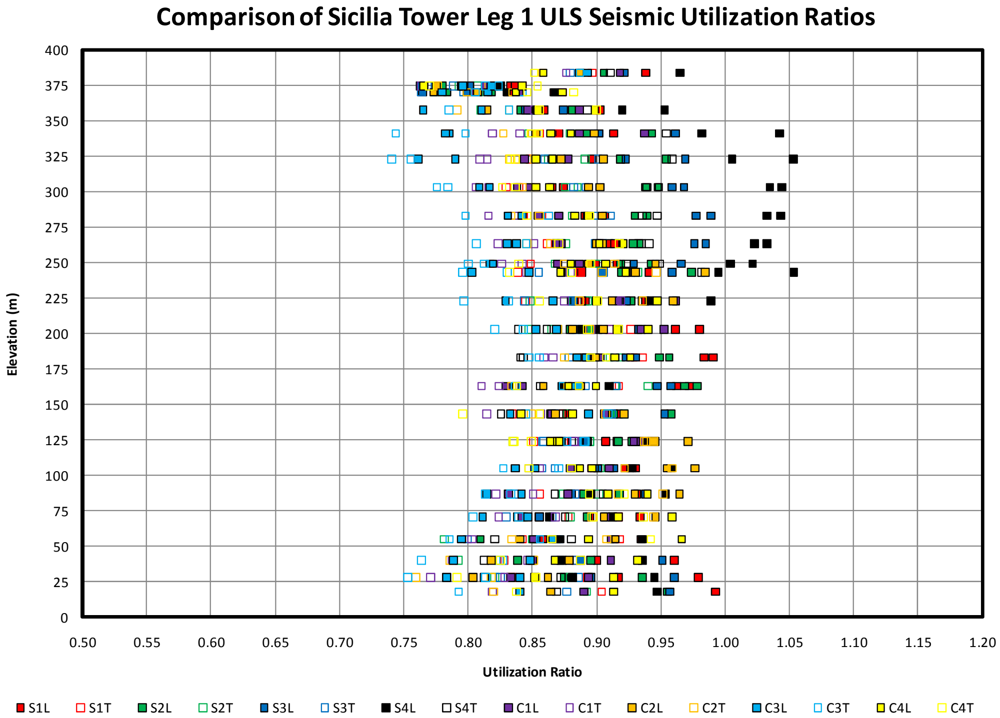
EL (m)	Time History Input – Longitudinal Combination - 1.0 x Long + 0.8 x Trans + 0.75 x Vert S1	S2	S3	S4	C1	C2	C3	C4	Transverse Combination - 0.8 x Long + 1.0 x Trans + 0.75 x Vert S1	S2	S3	S4	C1	C2	C3	C4	Maximum
18	0.99	0.92	0.96	0.91	0.93	0.91	0.86	0.96	0.92	0.85	0.90	0.87	0.85	0.86	0.78	0.88	0.99
28	0.92	0.85	0.89	0.86	0.89	0.85	0.80	0.90	0.86	0.79	0.84	0.82	0.81	0.80	0.74	0.83	0.92
28	0.98	0.91	0.96	0.92	0.95	0.91	0.85	0.96	0.92	0.85	0.90	0.88	0.87	0.86	0.79	0.89	0.98
40	0.89	0.83	0.88	0.88	0.90	0.84	0.78	0.89	0.85	0.78	0.84	0.85	0.83	0.79	0.75	0.83	0.90
40	0.95	0.89	0.94	0.94	0.97	0.90	0.84	0.95	0.91	0.84	0.90	0.91	0.90	0.85	0.81	0.89	0.97
55	0.84	0.80	0.86	0.88	0.90	0.86	0.80	0.86	0.82	0.77	0.83	0.86	0.84	0.81	0.76	0.81	0.90
55	0.90	0.86	0.92	0.94	0.97	0.92	0.86	0.92	0.88	0.84	0.89	0.92	0.90	0.87	0.83	0.87	0.97
71	0.86	0.84	0.88	0.87	0.89	0.84	0.81	0.85	0.83	0.82	0.85	0.84	0.84	0.81	0.81	0.81	0.89
71	0.90	0.88	0.92	0.93	0.97	0.92	0.87	0.93	0.88	0.86	0.91	0.92	0.91	0.88	0.85	0.87	0.97
87	0.89	0.85	0.89	0.88	0.88	0.88	0.80	0.89	0.87	0.84	0.88	0.84	0.84	0.85	0.83	0.85	0.89
87	0.94	0.88	0.95	0.94	0.94	0.94	0.86	0.95	0.92	0.87	0.94	0.90	0.89	0.90	0.84	0.90	0.95
105	0.94	0.89	0.92	0.91	0.92	0.89	0.88	0.92	0.92	0.87	0.89	0.88	0.87	0.87	0.86	0.90	0.94
105	0.95	0.89	0.93	0.94	0.95	0.92	0.89	0.95	0.90	0.87	0.91	0.90	0.89	0.88	0.85	0.90	0.95
123.65	0.95	0.89	0.89	0.91	0.94	0.91	0.88	0.90	0.91	0.89	0.87	0.89	0.90	0.87	0.84	0.90	0.95
123.65	0.95	0.91	0.86	0.92	0.92	0.91	0.87	0.88	0.89	0.87	0.82	0.87	0.87	0.89	0.89	0.84	0.95
124	0.95	0.91	0.86	0.92	0.91	0.91	0.87	0.88	0.89	0.87	0.82	0.87	0.87	0.89	0.89	0.84	0.95
124	0.95	0.91	0.86	0.92	0.92	0.91	0.87	0.88	0.89	0.87	0.82	0.87	0.87	0.89	0.89	0.84	0.95
143	0.91	0.88	0.87	0.84	0.85	0.84	0.83	0.85	0.86	0.83	0.83	0.79	0.82	0.80	0.83	0.81	0.91
143	0.95	0.92	0.92	0.88	0.89	0.88	0.88	0.89	0.90	0.88	0.87	0.84	0.88	0.85	0.89	0.85	0.95
163	0.95	0.93	0.92	0.88	0.88	0.88	0.86	0.89	0.90	0.87	0.85	0.82	0.86	0.83	0.83	0.84	0.95
163	0.93	0.94	0.92	0.84	0.91	0.90	0.87	0.89	0.92	0.89	0.87	0.83	0.90	0.85	0.87	0.84	0.94
183	0.95	0.94	0.88	0.87	0.91	0.89	0.89	0.91	0.94	0.93	0.86	0.84	0.88	0.86	0.87	0.87	0.95
183	0.96	0.95	0.89	0.87	0.91	0.90	0.89	0.92	0.95	0.94	0.87	0.84	0.89	0.87	0.88	0.87	0.96
203	0.95	0.93	0.90	0.88	0.96	0.91	0.89	0.92	0.95	0.93	0.87	0.88	0.91	0.87	0.88	0.88	0.96
203	0.94	0.92	0.88	0.86	0.94	0.89	0.87	0.91	0.93	0.91	0.86	0.86	0.89	0.86	0.85	0.86	0.94
223	0.91	0.91	0.92	0.91	0.97	0.93	0.90	0.89	0.90	0.87	0.91	0.91	0.91	0.92	0.86	0.85	0.97
223	0.87	0.86	0.88	0.87	0.92	0.89	0.86	0.85	0.86	0.83	0.87	0.87	0.87	0.87	0.82	0.81	0.92
243	0.93	0.96	0.95	0.93	**1.03**	0.95	0.87	0.94	0.91	0.92	0.91	0.91	0.88	0.92	0.85	0.86	**1.03**
243	0.88	0.90	0.89	0.88	0.97	0.89	0.82	0.88	0.86	0.87	0.86	0.88	0.83	0.87	0.82	0.81	0.97
249.048	0.90	0.93	0.91	0.90	0.97	0.91	0.83	0.89	0.87	0.89	0.86	0.89	0.84	0.87	0.83	0.82	0.97
249.048	0.93	0.90	0.91	0.98	0.89	0.90	0.83	0.89	0.90	0.88	0.85	0.93	0.84	0.84	0.88	0.83	0.98
263	0.95	0.93	0.93	**1.01**	0.90	0.92	0.85	0.90	0.92	0.90	0.88	0.91	0.84	0.86	0.84	0.84	**1.01**
263	0.95	0.92	0.92	**1.00**	0.89	0.92	0.84	0.88	0.94	0.90	0.87	0.96	0.83	0.85	0.89	0.85	**1.00**
283	0.96	0.94	0.93	**1.02**	0.87	0.93	0.84	0.88	0.91	0.91	0.89	0.87	0.82	0.86	0.79	0.83	**1.02**
283	0.97	0.95	0.94	**1.03**	0.88	0.94	0.85	0.89	0.93	0.92	0.89	0.93	0.84	0.87	0.88	0.85	**1.03**
303	0.96	0.93	0.98	**1.02**	0.88	0.92	0.83	0.87	0.91	0.90	0.88	0.82	0.82	0.85	0.78	0.81	**1.02**
303	0.97	0.94	0.99	**1.03**	0.89	0.93	0.84	0.88	0.92	0.91	0.89	0.82	0.83	0.86	0.79	0.82	**1.03**
323	0.93	0.89	0.96	0.99	0.81	0.88	0.80	0.82	0.88	0.87	0.88	0.79	0.80	0.83	0.77	0.79	0.99
323	0.97	0.93	1.00	**1.04**	0.84	0.92	0.83	0.86	0.92	0.90	0.92	0.80	0.84	0.86	0.77	0.81	**1.04**
341	0.91	0.87	0.93	0.96	0.83	0.85	0.83	0.82	0.87	0.84	0.87	0.87	0.80	0.80	0.86	0.80	0.96
341	0.96	0.92	0.98	**1.03**	0.87	0.90	0.80	0.83	0.92	0.88	0.92	0.85	0.82	0.84	0.79	0.79	**1.03**
357.4	0.91	0.84	0.90	0.92	0.84	0.81	0.85	0.84	0.93	0.79	0.86	0.95	0.83	0.80	0.88	0.83	0.95
357.4	0.87	0.83	0.87	0.88	0.80	0.79	0.80	0.80	0.88	0.77	0.82	0.90	0.79	0.76	0.83	0.79	0.90
369.9	0.88	0.77	0.82	0.92	0.80	0.77	0.85	0.82	0.90	0.79	0.82	0.96	0.82	0.78	0.89	0.82	0.96
369.9	0.84	0.74	0.79	0.89	0.77	0.74	0.81	0.78	0.87	0.76	0.78	0.92	0.79	0.75	0.85	0.79	0.92
374.447	0.81	0.75	0.78	0.90	0.77	0.76	0.83	0.79	0.87	0.78	0.79	0.94	0.80	0.76	0.87	0.80	0.94
374.447	0.82	0.80	0.86	0.86	0.83	0.79	0.77	0.82	0.79	0.77	0.83	0.83	0.82	0.77	0.78	0.81	0.86
383.619	0.93	0.89	0.95	0.98	0.97	0.91	0.90	0.95	0.93	0.90	0.96	0.98	0.94	0.89	0.89	0.94	0.98
383.619	0.93	0.89	0.95	0.98	0.97	0.91	0.90	0.95	0.93	0.90	0.96	0.98	0.94	0.89	0.89	0.94	0.98
Average	0.92	0.89	0.91	0.92	0.90	0.89	0.85	0.89	0.90	0.86	0.87	0.88	0.86	0.85	0.84	0.84	0.96
Maximum	0.99	0.96	1.00	1.04	1.03	0.95	0.90	0.96	0.95	0.94	0.96	0.98	0.94	0.92	0.89	0.94	1.04
Minimum	0.81	0.74	0.78	0.84	0.77	0.74	0.77	0.78	0.79	0.76	0.78	0.79	0.79	0.75	0.74	0.79	0.86

Calabria Tower Leg 1

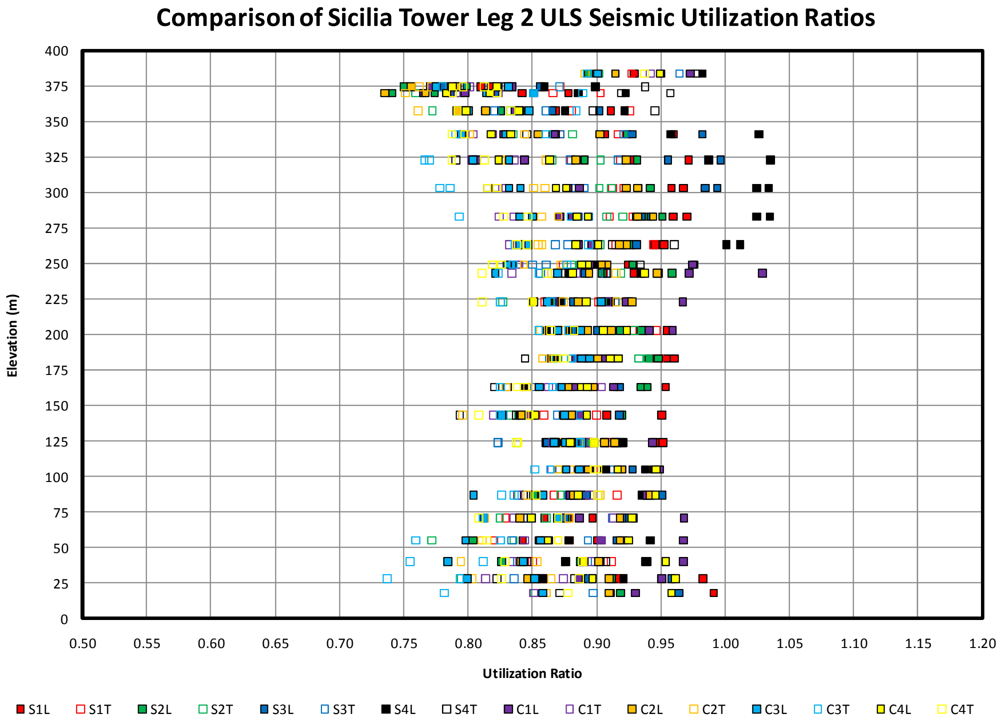
EL (m)	Time History Input																Maximum
	Longitudinal Combination - 1.0 x Long + 0.8 x Trans + 0.75 x Vert								Transverse Combination - 0.8 x Long + 1.0 x Trans + 0.75 x Vert								
	S1	S2	S3	S4	C1	C2	C3	C4	S1	S2	S3	S4	C1	C2	C3	C4	
18	0.96	0.93	**1.00**	0.88	0.89	0.89	0.85	0.94	0.89	0.90	0.93	0.84	0.83	0.82	0.80	0.89	**1.00**
28	0.92	0.87	0.93	0.82	0.84	0.83	0.81	0.89	0.85	0.83	0.86	0.78	0.78	0.78	0.78	0.85	0.93
28	0.98	0.93	0.99	0.88	0.90	0.89	0.87	0.96	0.91	0.90	0.93	0.83	0.84	0.83	0.84	0.91	0.99
40	0.92	0.85	0.89	0.83	0.84	0.82	0.81	0.91	0.86	0.82	0.84	0.78	0.78	0.79	0.77	0.86	0.92
40	**1.00**	0.93	0.97	0.90	0.91	0.88	0.87	0.98	0.93	0.89	0.91	0.84	0.85	0.86	0.84	0.94	**1.00**
55	0.93	0.85	0.85	0.84	0.85	0.84	0.82	0.91	0.89	0.82	0.80	0.79	0.80	0.83	0.77	0.88	0.93
55	**1.01**	0.91	0.91	0.90	0.90	0.91	0.89	0.98	0.97	0.89	0.86	0.87	0.86	0.89	0.83	0.95	**1.01**
71	0.96	0.86	0.85	0.87	0.86	0.87	0.85	0.92	0.92	0.86	0.82	0.83	0.85	0.88	0.81	0.91	0.96
71	0.99	0.88	0.90	0.90	0.90	0.90	0.88	0.95	0.95	0.88	0.85	0.85	0.87	0.92	0.83	0.93	0.99
87	0.92	0.86	0.88	0.87	0.85	0.88	0.84	0.88	0.90	0.86	0.86	0.84	0.86	0.89	0.85	0.87	0.92
87	0.97	0.90	0.95	0.93	0.91	0.93	0.87	0.93	0.94	0.90	0.91	0.90	0.89	0.93	0.89	0.91	0.97
105	0.89	0.90	0.95	0.89	0.89	0.93	0.86	0.91	0.88	0.88	0.92	0.86	0.86	0.92	0.91	0.91	0.95
105	0.90	0.89	0.99	0.93	0.91	0.92	0.83	0.85	0.86	0.85	0.94	0.90	0.86	0.90	0.83	0.83	0.99
123.65	0.86	0.86	0.98	0.89	0.88	0.93	0.89	0.86	0.84	0.86	0.92	0.86	0.85	0.92	0.88	0.85	0.98
123.65	0.85	0.90	1.00	0.93	0.92	0.94	0.87	0.89	0.82	0.89	0.95	0.91	0.89	0.94	0.86	0.88	1.00
124	0.85	0.90	1.00	0.93	0.92	0.94	0.87	0.89	0.83	0.89	0.95	0.91	0.89	0.94	0.86	0.88	1.00
124	0.85	0.90	1.00	0.93	0.92	0.94	0.87	0.89	0.83	0.89	0.95	0.91	0.89	0.94	0.86	0.87	1.00
143	0.92	0.88	0.97	0.86	0.86	0.90	0.85	0.87	0.88	0.85	0.93	0.84	0.82	0.87	0.84	0.87	0.97
143	0.94	0.89	0.93	0.86	0.85	0.90	0.86	0.89	0.91	0.86	0.92	0.84	0.86	0.89	0.86	0.89	0.94
163	0.96	0.90	0.92	0.84	0.85	0.91	0.88	0.88	0.93	0.88	0.90	0.83	0.83	0.89	0.86	0.87	0.96
163	0.96	0.90	0.93	0.85	0.86	0.91	0.88	0.88	0.93	0.88	0.90	0.84	0.85	0.91	0.87	0.89	0.96
183	0.99	0.91	0.91	0.88	0.88	0.91	0.89	0.88	0.94	0.88	0.88	0.83	0.86	0.90	0.86	0.88	0.99
183	0.97	0.90	0.89	0.86	0.88	0.91	0.88	0.88	0.94	0.88	0.87	0.82	0.86	0.90	0.86	0.87	0.97
203	0.99	0.91	0.91	0.89	0.90	0.90	0.88	0.90	0.94	0.88	0.88	0.83	0.87	0.89	0.86	0.89	0.99
203	0.98	0.89	0.89	0.88	0.89	0.89	0.86	0.88	0.93	0.87	0.87	0.82	0.86	0.88	0.84	0.87	0.98
223	0.98	0.89	0.95	0.91	0.92	0.88	0.87	0.90	0.92	0.86	0.90	0.88	0.88	0.85	0.84	0.88	0.98
223	0.95	0.86	0.92	0.89	0.90	0.86	0.84	0.87	0.89	0.83	0.88	0.85	0.85	0.81	0.81	0.84	0.95
243	0.93	0.89	0.98	0.93	0.93	0.89	0.86	0.91	0.87	0.86	0.92	0.88	0.88	0.84	0.83	0.87	0.98
243	0.90	0.86	0.95	0.90	0.90	0.86	0.84	0.88	0.85	0.83	0.89	0.86	0.85	0.81	0.80	0.85	0.95
249.048	0.89	0.87	0.97	0.90	0.92	0.86	0.84	0.89	0.84	0.84	0.91	0.86	0.86	0.81	0.80	0.85	0.97
249.048	0.90	0.87	0.99	0.90	0.93	0.91	0.82	0.90	0.85	0.84	0.93	0.86	0.87	0.89	0.79	0.87	0.99
263	0.86	0.89	**1.02**	0.92	0.94	0.90	0.81	0.89	0.82	0.85	0.95	0.86	0.87	0.85	0.78	0.86	**1.02**
263	0.90	0.89	**1.02**	0.91	0.94	0.91	0.85	0.90	0.87	0.87	0.95	0.87	0.88	0.91	0.83	0.89	**1.02**
283	0.86	0.89	**1.03**	0.92	0.92	0.88	0.84	0.88	0.83	0.85	0.95	0.86	0.86	0.85	0.83	0.86	**1.03**
283	0.87	0.90	**1.04**	0.92	0.93	0.89	0.86	0.89	0.85	0.86	0.96	0.87	0.87	0.86	0.84	0.87	**1.04**
303	0.85	0.87	**1.03**	0.90	0.90	0.87	0.84	0.87	0.82	0.85	0.95	0.85	0.85	0.84	0.83	0.85	**1.03**
303	0.86	0.91	**1.07**	0.93	0.94	0.89	0.84	0.88	0.82	0.85	0.98	0.87	0.87	0.84	0.82	0.84	**1.07**
323	0.82	0.87	**1.03**	0.88	0.90	0.87	0.82	0.86	0.79	0.82	0.96	0.83	0.85	0.84	0.80	0.83	**1.03**
323	0.85	0.90	**1.07**	0.92	0.93	0.89	0.83	0.89	0.80	0.84	0.99	0.86	0.87	0.85	0.80	0.85	**1.07**
341	0.82	0.86	0.99	0.85	0.87	0.83	0.82	0.87	0.81	0.81	0.93	0.83	0.83	0.85	0.81	0.86	0.99
341	0.86	0.92	**1.05**	0.90	0.92	0.86	0.84	0.88	0.81	0.86	0.98	0.84	0.86	0.82	0.83	0.87	**1.05**
357.4	0.82	0.87	0.95	0.85	0.86	0.86	0.82	0.88	0.81	0.84	0.90	0.84	0.83	0.88	0.83	0.88	0.95
357.4	0.82	0.91	**1.02**	0.86	0.89	0.85	0.82	0.87	0.77	0.85	0.95	0.81	0.84	0.82	0.81	0.86	**1.02**
369.9	0.77	0.86	0.93	0.81	0.81	0.84	0.80	0.84	0.79	0.82	0.88	0.78	0.81	0.87	0.79	0.84	0.93
369.9	0.75	0.77	0.81	0.76	0.77	0.82	0.74	0.81	0.77	0.79	0.80	0.75	0.80	0.86	0.76	0.82	0.86
374.447	0.76	0.78	0.80	0.76	0.78	0.84	0.75	0.81	0.79	0.81	0.82	0.75	0.81	0.88	0.78	0.84	0.88
374.447	0.79	0.80	0.82	0.80	0.83	0.80	0.80	0.82	0.78	0.80	0.80	0.79	0.83	0.80	0.76	0.82	0.83
383.619	0.90	0.91	0.95	0.90	0.93	0.90	0.89	0.92	0.89	0.89	0.91	0.89	0.93	0.89	0.89	0.91	0.95
383.619	0.90	0.91	0.95	0.90	0.93	0.90	0.89	0.92	0.89	0.89	0.91	0.89	0.93	0.89	0.89	0.91	0.95
Average	0.90	0.88	0.95	0.88	0.89	0.88	0.85	0.89	0.86	0.86	0.91	0.85	0.85	0.87	0.83	0.87	0.98
Maximum	1.01	0.93	1.07	0.93	0.94	0.94	0.89	0.98	0.97	0.90	0.99	0.91	0.93	0.94	0.91	0.95	1.07
Minimum	0.75	0.77	0.80	0.76	0.77	0.80	0.74	0.81	0.77	0.79	0.80	0.75	0.78	0.78	0.76	0.82	0.83

Calabria Tower Leg 2

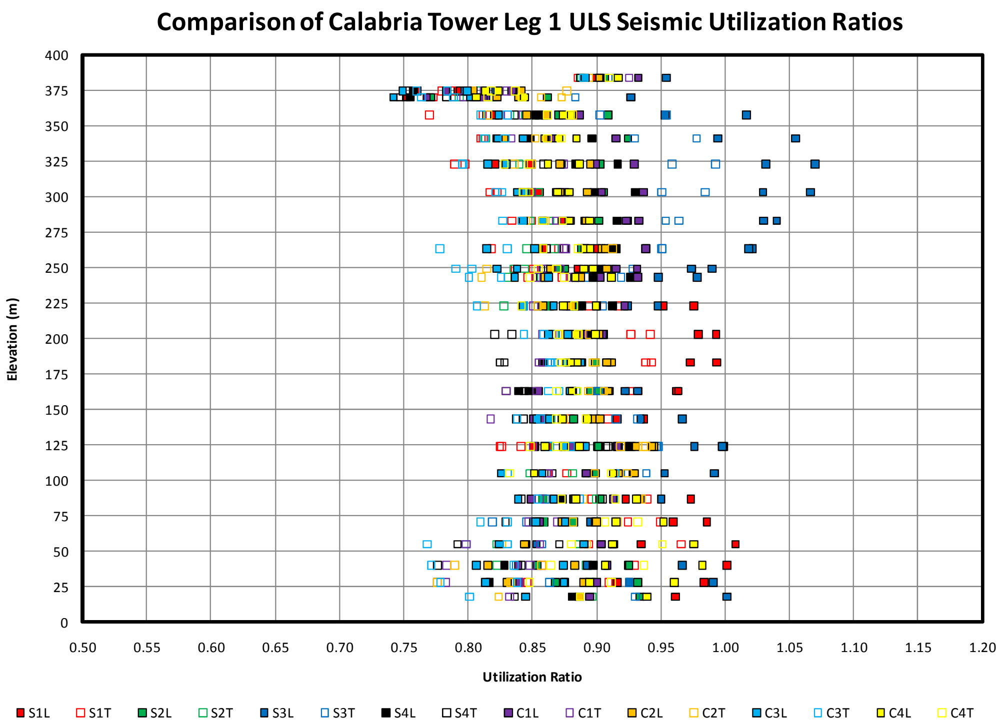
EL (m)	Time History Input																Maximum
	Longitudinal Combination - 1.0 x Long + 0.8 x Trans + 0.75 x Vert								Transverse Combination - 0.8 x Long + 1.0 x Trans + 0.75 x Vert								
	S1	*S2*	*S3*	*S4*	*C1*	*C2*	*C3*	*C4*	*S1*	*S2*	*S3*	*S4*	*C1*	*C2*	*C3*	*C4*	
18	0.95	0.94	1.00	0.87	0.90	0.89	0.82	0.90	0.89	0.90	0.92	0.80	0.83	0.83	0.79	0.84	1.00
28	0.90	0.87	0.92	0.85	0.85	0.83	0.80	0.86	0.83	0.84	0.86	0.78	0.79	0.78	0.75	0.80	0.92
28	0.97	0.94	0.99	0.91	0.91	0.89	0.86	0.92	0.89	0.90	0.92	0.84	0.85	0.84	0.82	0.86	0.99
40	0.91	0.85	0.90	0.86	0.84	0.82	0.80	0.86	0.85	0.82	0.84	0.81	0.79	0.77	0.78	0.81	0.91
40	0.99	0.92	0.97	0.93	0.91	0.89	0.87	0.93	0.92	0.89	0.91	0.88	0.85	0.84	0.85	0.87	0.99
55	0.93	0.83	0.86	0.88	0.87	0.85	0.83	0.86	0.88	0.80	0.81	0.85	0.84	0.82	0.80	0.81	0.93
55	0.99	0.88	0.92	0.94	0.94	0.91	0.89	0.92	0.95	0.86	0.87	0.92	0.91	0.89	0.87	0.87	0.99
71	0.94	0.85	0.89	0.90	0.90	0.88	0.86	0.84	0.91	0.83	0.87	0.88	0.88	0.86	0.84	0.82	0.94
71	0.98	0.87	0.92	0.93	0.93	0.91	0.88	0.89	0.93	0.86	0.89	0.90	0.90	0.88	0.87	0.85	0.98
87	0.91	0.85	0.86	0.85	0.86	0.87	0.84	0.84	0.88	0.83	0.85	0.84	0.83	0.85	0.83	0.83	0.91
87	0.97	0.90	0.94	0.91	0.91	0.92	0.89	0.89	0.93	0.87	0.90	0.88	0.87	0.89	0.87	0.88	0.97
105	0.91	0.90	0.92	0.90	0.90	0.87	0.87	0.88	0.90	0.88	0.90	0.89	0.87	0.87	0.88	0.88	0.92
105	0.91	0.90	0.96	0.89	0.91	0.86	0.82	0.88	0.87	0.85	0.91	0.85	0.86	0.83	0.81	0.83	0.96
123.65	0.88	0.86	0.95	0.87	0.88	0.83	0.86	0.84	0.88	0.86	0.89	0.91	0.85	0.84	0.88	0.85	0.95
123.65	0.88	0.89	0.98	0.90	0.89	0.91	0.87	0.86	0.85	0.86	0.92	0.87	0.86	0.89	0.86	0.86	0.98
124	0.88	0.89	0.98	0.90	0.89	0.90	0.87	0.86	0.85	0.86	0.92	0.87	0.86	0.89	0.86	0.86	0.98
124	0.88	0.89	0.98	0.90	0.89	0.90	0.87	0.86	0.85	0.86	0.92	0.87	0.86	0.89	0.86	0.86	0.98
143	0.90	0.87	0.96	0.83	0.87	0.86	0.85	0.86	0.86	0.83	0.91	0.82	0.84	0.83	0.84	0.84	0.96
143	0.91	0.86	0.92	0.85	0.86	0.87	0.86	0.87	0.88	0.85	0.88	0.84	0.84	0.85	0.85	0.86	0.92
163	0.94	0.88	0.93	0.88	0.92	0.85	0.87	0.84	0.90	0.86	0.88	0.86	0.91	0.83	0.85	0.84	0.94
163	0.94	0.88	0.93	0.88	0.93	0.85	0.86	0.84	0.91	0.87	0.88	0.87	0.92	0.85	0.85	0.85	0.94
183	0.98	0.87	0.92	0.91	0.93	0.86	0.87	0.86	0.93	0.87	0.88	0.88	0.90	0.84	0.86	0.84	0.98
183	0.96	0.85	0.90	0.90	0.92	0.85	0.87	0.86	0.92	0.87	0.86	0.87	0.90	0.84	0.85	0.84	0.96
203	0.98	0.86	0.89	0.92	0.91	0.88	0.87	0.88	0.93	0.85	0.87	0.88	0.90	0.86	0.85	0.84	0.98
203	0.97	0.85	0.88	0.91	0.90	0.87	0.85	0.87	0.91	0.83	0.85	0.87	0.88	0.85	0.83	0.83	0.97
223	0.97	0.86	0.92	0.91	0.93	0.89	0.87	0.89	0.91	0.86	0.88	0.87	0.89	0.86	0.85	0.85	0.97
223	0.95	0.82	0.90	0.89	0.91	0.87	0.85	0.87	0.89	0.80	0.85	0.85	0.87	0.83	0.82	0.83	0.95
243	0.92	0.87	0.97	0.91	0.94	0.90	0.87	0.87	0.87	0.84	0.90	0.87	0.90	0.86	0.84	0.81	0.97
243	0.89	0.85	0.94	0.88	0.91	0.87	0.85	0.84	0.84	0.82	0.88	0.84	0.87	0.83	0.81	0.78	0.94
249.048	0.88	0.86	0.96	0.89	0.92	0.88	0.85	0.88	0.83	0.83	0.90	0.85	0.88	0.83	0.82	0.80	0.96
249.048	0.88	0.84	0.97	0.87	0.91	0.88	0.81	0.86	0.83	0.81	0.91	0.85	0.89	0.83	0.82	0.83	0.97
263	0.84	0.86	**1.01**	0.89	0.92	0.88	0.81	0.86	0.80	0.81	0.94	0.84	0.86	0.83	0.79	0.82	**1.01**
263	0.89	0.88	**1.00**	0.89	0.92	0.88	0.85	0.86	0.86	0.88	0.94	0.87	0.91	0.84	0.84	0.84	**1.00**
283	0.85	0.88	**1.03**	0.90	0.92	0.86	0.83	0.87	0.83	0.83	0.95	0.84	0.85	0.82	0.81	0.82	**1.03**
283	0.87	0.89	**1.04**	0.91	0.93	0.87	0.85	0.88	0.84	0.85	0.96	0.85	0.86	0.84	0.83	0.84	**1.04**
303	0.84	0.88	**1.03**	0.88	0.90	0.86	0.83	0.87	0.81	0.83	0.95	0.83	0.84	0.83	0.81	0.82	**1.03**
303	0.85	0.91	**1.06**	0.92	0.93	0.87	0.84	0.88	0.81	0.86	0.98	0.85	0.87	0.83	0.81	0.82	**1.06**
323	0.84	0.88	**1.01**	0.86	0.88	0.84	0.81	0.84	0.81	0.83	0.93	0.81	0.82	0.81	0.79	0.80	**1.01**
323	0.87	0.91	**1.05**	0.89	0.91	0.86	0.84	0.87	0.82	0.85	0.97	0.82	0.85	0.82	0.80	0.81	**1.05**
341	0.86	0.86	0.96	0.86	0.89	0.85	0.81	0.82	0.85	0.82	0.89	0.85	0.88	0.83	0.84	0.80	0.96
341	0.88	0.91	**1.06**	0.88	0.91	0.87	0.84	0.88	0.86	0.84	0.98	0.85	0.88	0.84	0.81	0.82	**1.06**
357.4	0.87	0.85	0.93	0.87	0.91	0.85	0.82	0.84	0.86	0.86	0.87	0.92	0.92	0.83	0.86	0.83	0.93
357.4	0.89	0.89	0.98	0.86	0.90	0.85	0.82	0.85	0.85	0.83	0.91	0.86	0.87	0.81	0.80	0.80	0.98
369.9	0.85	0.83	0.89	0.88	0.88	0.81	0.80	0.81	0.82	0.83	0.83	0.92	0.89	0.79	0.85	0.80	0.92
369.9	0.80	0.79	0.78	0.86	0.85	0.76	0.79	0.77	0.80	0.81	0.79	0.90	0.86	0.75	0.83	0.77	0.90
374.447	0.80	0.81	0.79	0.88	0.86	0.76	0.80	0.78	0.81	0.83	0.80	0.93	0.88	0.77	0.85	0.79	0.93
374.447	0.80	0.83	0.83	0.80	0.80	0.82	0.80	0.82	0.80	0.82	0.80	0.79	0.78	0.82	0.79	0.80	0.83
383.619	0.89	0.93	0.96	0.90	0.90	0.91	0.89	0.91	0.89	0.91	0.92	0.89	0.88	0.92	0.88	0.89	0.96
383.619	0.89	0.93	0.96	0.90	0.90	0.91	0.89	0.91	0.89	0.91	0.92	0.89	0.88	0.92	0.88	0.89	0.96
Average	0.90	0.87	0.94	0.89	0.90	0.87	0.85	0.86	0.87	0.85	0.89	0.86	0.87	0.84	0.83	0.83	0.97
Maximum	0.99	0.94	1.06	0.94	0.94	0.92	0.89	0.93	0.95	0.91	0.98	0.93	0.92	0.92	0.88	0.89	1.06
Minimum	0.80	0.79	0.78	0.80	0.80	0.76	0.79	0.77	0.80	0.80	0.79	0.78	0.78	0.75	0.75	0.77	0.83

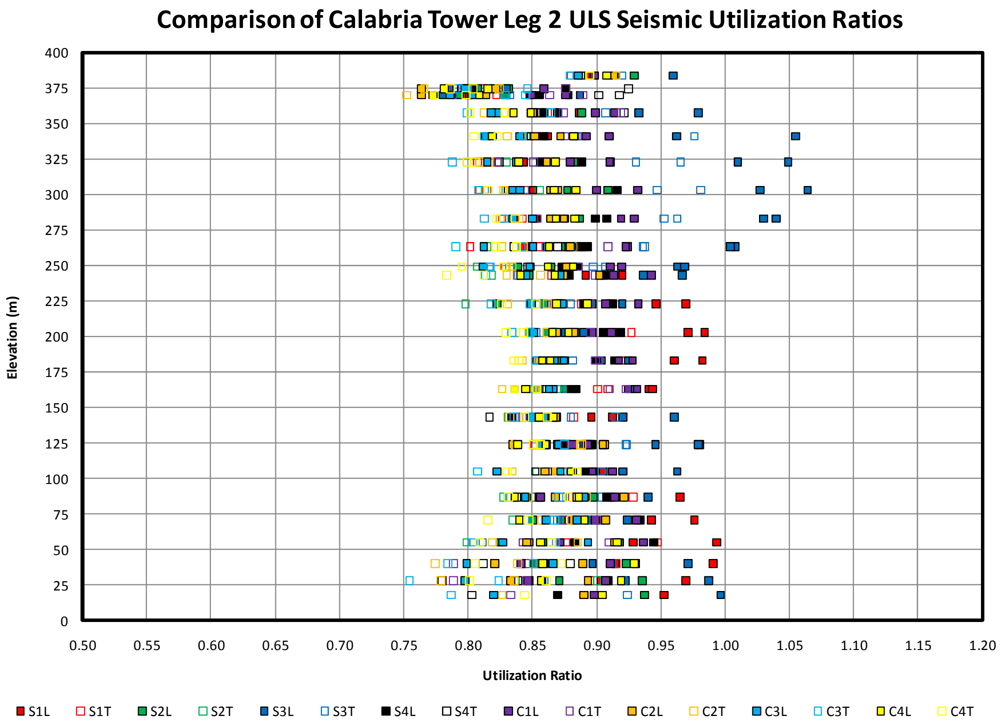


Comparison of Sicilia Tower Leg 1 ULS Seismic Utilization Ratios
Elevation (m)
400
375
350
325
300
275
250
225
200
175
150
125
100
75
50
25
0
0.50 0.55 0.60 0.65 0.70 0.75 0.80 0.85 0.90 0.95 1.00 1.05 1.10 1.15 1.20
Utilization Ratio
S1L S1T S2L S2T S3L S3T S4L S4T C1L C1T C2L C2T C3L C3T C4L C4T



Comparison of Sicilia Tower Leg 2 ULS Seismic Utilization Ratios
Elevation (m)
400
375
350
325
300
275
250
225
200
175
150
125
100
75
50
25
0
0.50 0.55 0.60 0.65 0.70 0.75 0.80 0.85 0.90 0.95 1.00 1.05 1.10 1.15 1.20
Utilization Ratio
S1L S1T S2L S2T S3L S3T S4L S4T C1L C1T C2L C2T C3L C3T C4L C4T


Comparison of Calabria Tower Leg 1 ULS Seismic Utilization Ratios
Elevation (m)
Utilization Ratio
0.50 0.55 0.60 0.65 0.70 0.75 0.80 0.85 0.90 0.95 1.00 1.05 1.10 1.15 1.20
0 25 50 75 100 125 150 175 200 225 250 275 300 325 350 375 400
S1L S1T S2L S2T S3L S3T S4L S4T C1L C1T C2L C2T C3L C3T C4L C4T


Comparison of Calabria Tower Leg 2 ULS Seismic Utilization Ratios
Elevation (m)
Utilization Ratio
0.50 0.55 0.60 0.65 0.70 0.75 0.80 0.85 0.90 0.95 1.00 1.05 1.10 1.15 1.20
0 25 50 75 100 125 150 175 200 225 250 275 300 325 350 375 400
S1L S1T S2L S2T S3L S3T S4L S4T C1L C1T C2L C2T C3L C3T C4L C4T

5.1.1.9 Shear Stresses

FOR LONGITUDINAL SHEAR THE MAXIMUM SHEAR STRESS ANYWHERE IN THE TOWER IS $\tau_{MAX} = 43.4$ MPA

BASED ON MODEL VERSION 3.3. CHANGES SINCE ARE SMALL AND SHEAR STRESSES ARE STILL NOT CRITICAL.

THE CONTROLLING CAPACITY WILL BE PLATE D WHICH HAS THE LARGEST WIDTH & THINNEST PLATE

THE MINIMUM SIZE OF PLATE D ANYWHERE IN THE TOWER IS

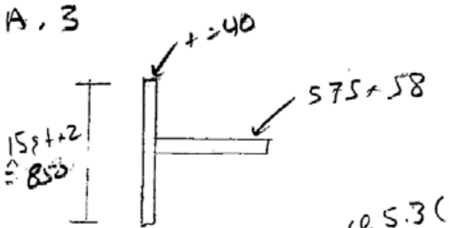
$t = 40$ mm

STIFFENERS = 575 × 58

DETERMINE SHEAR CAPACITY:

SHEAR BUCKLING OF THE ENTIRE PANEL CANNOT BE AN ISSUE SINCE THE PANEL IS DESIGN TO BE NEARLY FULLY EFFECTIVE UNDER LONG. STRESS. JUST TO PROVE THIS

FROM ANNEX A.3



$I_{sp} = 2.5$ E09

$\Sigma I_{sl} = 5 \times 2.5$ E09 $\times 1/3 = 4160 \times 10^6$ (A5.3(4))

$k_{\tau sl} = 9\left(\frac{8000}{3500}\right)^2 \sqrt[4]{\left(\frac{4160 \times 10^6}{40^3 \times 8000}\right)^3} = 226$

$k_\tau = 4 + 5.34\,(8000/3500)^2 + 226 = 257$

$\lambda_w = 8000 / (37.4 \times 40 \times .71)\sqrt{257} = 0.460 < 0.83$

∴ NO SHEAR BUCKLING OF PANEL

THEREFORE, THE SHEAR CAPACITY WILL BE GOVERNED BY SUB-PANEL BUCKLING BETWEEN STIFFENERS

THE MAX STIFFENER SPACING ON PLATE D = 1600mm

$$k_{\tau i} = 5.34 + 4\left(1600/3500\right)^2 = 6.2$$

$$\lambda_w = \frac{1600}{(37.4)(40)(0.71)\sqrt{6.2}} = 0.605 < 0.83$$

∴ NO SHEAR BUCKLING OF SUB-PANELS

THEREFORE SHEAR CAPACITY OF PLATE D IS:

$$\frac{\chi_w f_{yw}}{\sqrt{3}\,\gamma_{M1}} = \frac{(1.0)(460)}{\sqrt{3} \times 1.1} = 241 \text{ MPa}$$

COMPARED WITH MAX τ_{MAX} = 43 MPa

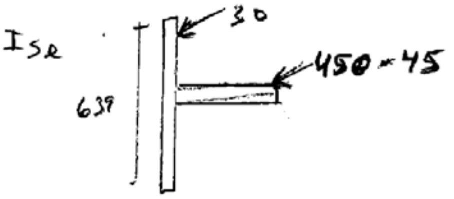
∴ SHEAR CAPACITY IS NOT AN ISSUE

SHEAR DEMAND IS MUCH LESS THAN SHEAR CAPACITY SO THE INTERACTION OF SHEAR WITH LONGITUDINAL STRESS DOES NOT NEED TO BE CONSIDERED AS PER EN 1993 1-5 7.1 (1)

PERFORM A SIMILAR INVESTIGATION FOR TRANSVERSE SHEAR

CONSIDERING PLATE G ($t_{min} = 30$, 450×45)

FOR 2 STIFF WITH $\alpha = a/h_w = 3500/4000 = 0.875 < 3$



$I_{se} = 9.1e8$

$\Sigma I_{se} = 2 \times 9.1e8 \times 1/3$
$= 606 \times 10^6$

$$k_\tau = 4.1 + \frac{6.3 + 0.18 \frac{606 \times 10^6}{(30^3)(4000)}}{0.875^2} + 2.2 \sqrt[3]{\frac{606 \times 10^6}{(30^3)(4000)}}$$

$$4.1 + 9.55 + 3.91 = 17.6$$

$$\lambda_w = \frac{4000}{(37.4)(30)\,0.71\sqrt{17.5}} = 1.20$$

$$\chi_w = 1.37/(0.7 + 1.20) \simeq 0.72$$

↑ SHEAR REDUCTION FOR BUCKLING OF ENTIRE PANEL

CHECK SUB-PANEL

$$k_\tau = 5.34 + 4\,(1600/3500)^2 = 6.18$$

$$\lambda_w = \frac{1600}{(37.4)(30)(0.71)(\sqrt{6.18})} = 0.81 < 0.83$$

∴ NO SUB PANE SHEAR BUCKLING

THEREFORE, THE MAXIMUM SHEAR CAPACITY FOR PLATE G IS

$$\frac{\chi_w f_y}{\sqrt{3} * \gamma_m} = \frac{(0.72)(460)}{\sqrt{3} * 1.1} \cong 175 \text{ MPa}$$

↑ COMPARED WITH MAX SHEAR STRESS OF 33 MPa

∴ SHEAR CAPACITY IS NOT AN ISSUE & SHEAR DEMAND IS LESS THAN 1/2 CAPACITY SO NO INTERACTION BETWEEN SHEAR & LONGITUDINAL STRESS IS REQUIRED

- CHECK FORCES ON TRANSVERSE STIFFENER DUE TO SHEAR BUCKLING

← EN 1993 1-5 9.3.3 (3)

$$\frac{1}{\bar{\lambda}_w^2} \, f_{yw}/\sqrt{3} * 1.1$$

$$0.69 \left(\frac{460}{\sqrt{3} * 1.1}\right) = 167 < 33 \text{ MPa}$$

∴ NO FORCES DUE TO TENSION FIELD

5.1.2 Joints and Splices

SUMMARY OF SPLICE DESIGN CALCULATIONS FOR TOWER SEGMENT TRANSVERSE SPLICES.
{ALL DETAILED CALCULATIONS DONE IN SPREADSHEETS → SAMPLE SPREADSHEET PRINTS ATTACHED AT THE END OF THESE NOTES... }

→ TOWER SEGMENT JOINTS CHECKED FOR THE FOLLOWING:

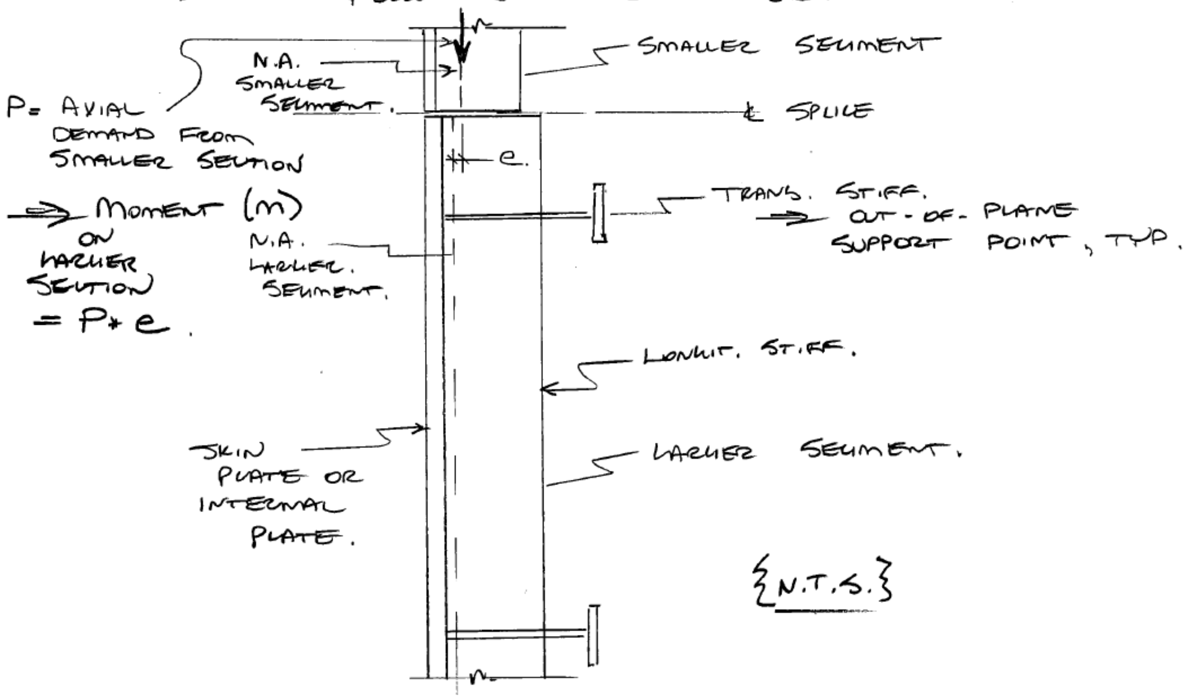
[1] — ECCENTRICITY OF TOWER PLATES OF DIFFERENT SIZES JOINING @ SPLICE (SKIN PL'S, INTERNAL PL'S AND LONGITUDINAL STIFFENERS).

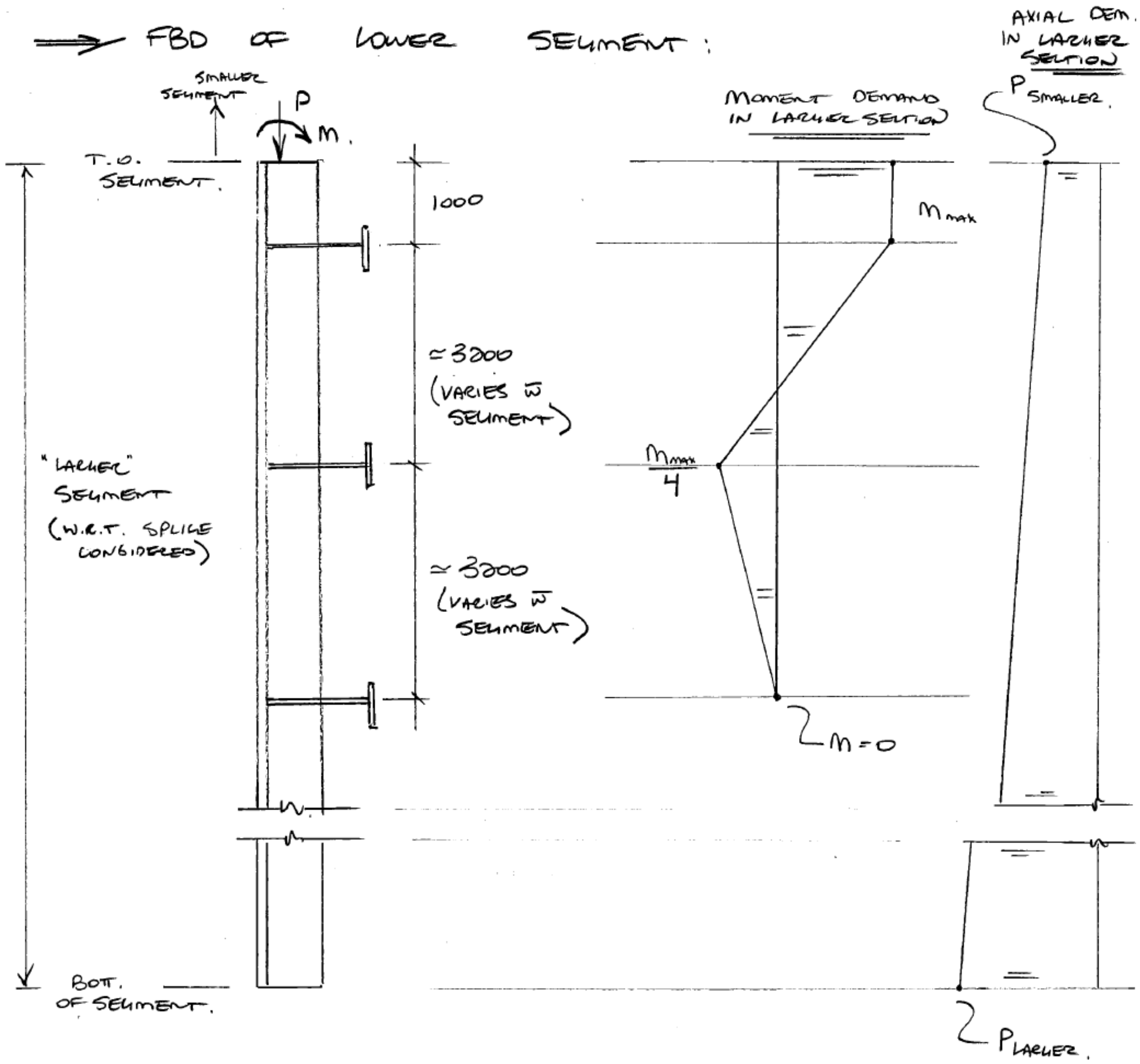
[2] — DESIGN OF LONGIT. STIFF BOLTED SPLICES.

[3] — DESIGN OF INTERNAL PL. BOLTED SPLICES
→ ALL SKIN PL'S WELDED USING FULL PENETRATION BUTT WELDS. ⇒ DEVELOPS YIELD STRENGTH OF PLATE.

[1] — ECCENTRICITY CHECK FOR PLATE SEGMENTS OF DIFFERENT SIZES JOINING AT SPLICE:

→ THE FOLLOWING WAS ASSUMED:





→ $P_{SMALLER}$ TAKEN AS COMPRESSIVE CAPACITY OF SMALLER SECTION SPLICED.

→ P_{LARGER} TAKEN AS " " " " LARGER SECTION SPLICED.

→ AXIAL "DEMAND" IN LARGER SECTION INTERPOLATED LINEARLY B/W $P_{SMALLER}$ TO P_{LARGER}.

→ ALL MOMENT FROM ECCENTRICITY IS ASSUMED TO BE TAKEN BY THE LARGER SECTION → IT PREVENTS THE SMALLER SECTION FROM DEFORMING.
{AS PER DISCUSSION WITH CMK.}

→ IT IS ALSO ASSUMED THAT THE MOMENT DIES OUT AFTER 2 "BAYS" DOWN — AS PER DISCUSSION WITH CMK.

→ WHEN COMPUTING THE ALLOWABLE STRESSES IN THE STEEL, FOR THE TOP (1m LONG) SECTION, IT IS ASSUMED THAT F_y GOVERNS (i.e. THERE IS NO REDUCTION FOR COLUMN BUCKLING OVER THIS SHORT LENGTH)

— FOR THE LOWER PANELS, THE REDUCED ALLOWABLE STRESS CONSIDERING COLUMN BUCKLING IS USED.

∴ A SAMPLE OF SECTIONS THAT WERE CHECKED CAN BE SEEN IN THE ATTACHED SPREADSHEET PRINTOUT...

[2] — DESIGN OF LONGIT. STIFF BOLTED SPLICES.

→ SPLICES DESIGNED AS SLIP CRITICAL AT ULS.
→ STIFFENERS SPLICED FOR DESIGN YIELD CAPACITY.

→ FOR 750 × 75 STIFF:

→ CAPACITY $= \dfrac{(750)(75)(460)(10^{-3})}{1.05 \; \gamma_{M0}} = 24643$ kN.

→ BOLT SLIP CAPACITY: EN 1993-1-8 § 3.9.1

→ $F_{p,c} = 0.7 f_{ub} A_s = 393$ kN / BOLT FOR M30 10.9 (FROM EN1090-2, TABLE 19).

→ $F_{s,Rd} = \dfrac{k_s n \mu}{\gamma_{M3}} F_{pc} = \dfrac{1(2)(0.45)}{1.25}(393)$

$= 283$ kN / BOLT. { SLIP FACTOR OF $\mu = 0.45$ HAS BEEN SPECIFIED }

→ MIN # OF BOLTS REQ'D = $\frac{24643 \text{ kN}}{283 \text{ kN/BOLT}}$

= 87.1 → SAY 88 BOLTS.

CHECK BOLT BEARING CAPACITY: {TABLE 3.4}

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$ → VARIES DEPENDING ON LOCATION — ALL BOLTS CHECKED IN SPREADSHEET (DOES NOT GOVERN).

→ SPLICE PLATE THICKNESS SIZED TO PROVIDE AT LEAST THE SAME AREA AS STIFF. BEING SPLICED.

→ TENSION ON NET SECTION:

→ NET SECTION CAPACITY {TAKE F_u = 540 MPa, γ_{M2} = 1.25}

$$N_{u,Rd} = \frac{0.9 F_u A_{net}}{\gamma_{M2}}$$ ← SPLICE PL'S = 45 x 640 w/ 8 ROWS OF BOLTS.

→ $A_{NET} = 2(45)[640 - 8(33)] = 33840 \text{ mm}^2$.

→ $N_{u,Rd} = \frac{0.9(540)(33840)(10^{-3})}{1.25} = 13157 \text{ kN}$.

→ THIS RESULTS IN A MAX ALLOWABLE TENSION STRESS IN THE STIFFENER OF:

$$\frac{13157 \times 10^3}{750(75)} = 234 \text{ MPa}$$

→ FROM TOWER STEEL DESIGN RESULTS, MAX TENSION STRESS ANYWHERE IN TOWER PLATES (VERTICAL) = 193 MPa < 234 → NET SECTION O.K. ✓

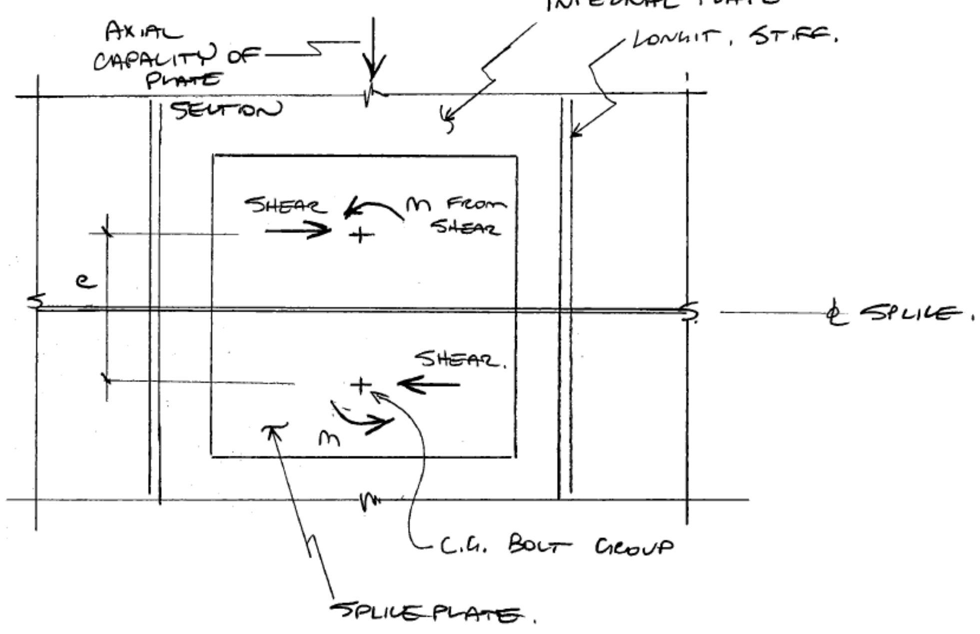
{SEE SUBSEQUENT CALC NOTES FOR THOROUGH NET SECTION CHECKS ...}

3 INTERNAL PLATE BOLTED SPLICES:

→ SECTIONS SPLICED FOR YIELD CAPACITY OF INTERNAL PLATE.

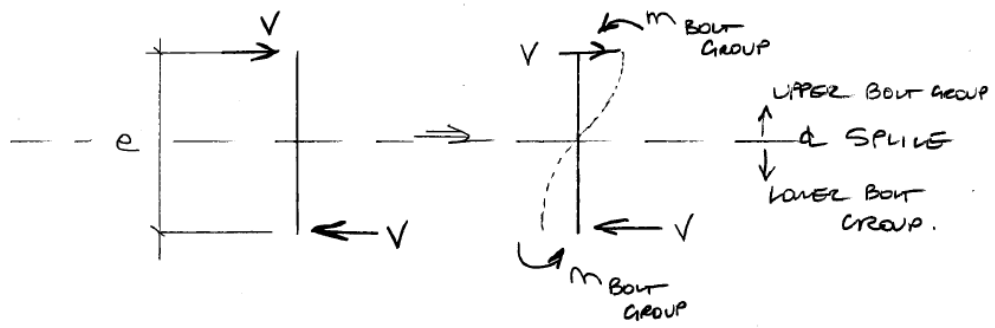
→ IN ADDITION, SHEAR + MOMENT FROM SHEAR WERE ALSO ADDED TO BOLT DEMANDS.

→ TYPICAL SECTION:

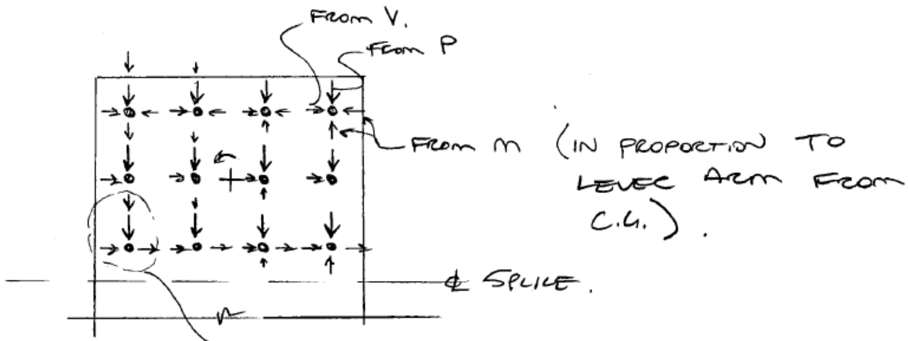


→ SPLICE PLATE MUST TRANSFER GLOBAL SHEAR DEMAND ACROSS THE SPLICE → ASSUME IT DOES SO FROM C.G. OF BOLT GROUPS

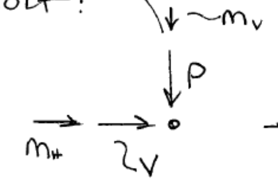
→ F.B.D. OF SPLICE PLATE:



→ FOR A SIMPLE BOLT GROUP, FORCES ARE RESISTED (COMPUTED) AS SHOWN:



→ WORST CASE BOLT:



$$\Rightarrow V_{BOLT} = \left[(m_H + V)^2 + (P + m_V)^2 \right]^{\frac{1}{2}}$$

→ BOLT LAYOUT IS CHOSEN SUCH THAT DEMAND ON WORST CASE BOLT DOES NOT EXCEED IT'S CAPACITY.

→ ALL CALCULATIONS DONE IN SPREADSHEET, USING ACTUAL GLOBAL SHEAR DEMANDS ON LEG AT SEGMENT CONSIDERED.

→ GLOBAL SHEAR DEMAND IS CONVERTED TO SHEAR STRESS (AVERAGE), THEN SHEAR STRESS IS CONVERTED BACK TO SHEAR FORCE ATTRIBUTED TO THE SEGMENT OF PLATE BEING SPLICED.

→ ALL CALCULATIONS DONE IN A SPREADSHEET WHICH ITERATES ON THE BOLT LAYOUT FOR EACH SPLICE UNTIL A SUFFICIENT # OF BOLTS IS CHOSEN TO RESIST ALL LOADS + EFFECTS.

→ SAMPLE INPUT SHEET FOR ONE SPLICE IS ATTACHED AT THE END OF THESE CALC. NOTES...

→ SPLICE PLATES WERE SIZED FOR COMBINED DEMANDS IN COMPRESSION AND SHEAR:

→ COMPRESSIVE LOAD – BASED ON DESIGN YIELD CAPACITY OF SECTION BEING SPLICED

→ SHEAR DEMAND – COMPUTED AS DESCRIBED ON PREVIOUS PAGE...

→ TOTAL STRESS DEMAND ON SECTION COMPUTED USING VON MISES CRITERIA:

$$\Rightarrow \sigma_{VM} = \sqrt{\sigma_{Comp}^2 + 3(\sigma_V)^2}$$

→ ALL SPLICE PLATES SIZED USING SPREADSHEET...

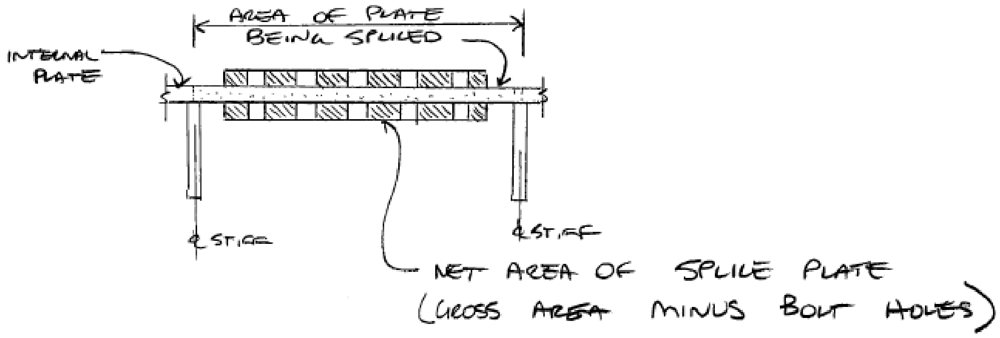
→ FOR CLEARANCES ON SPLICE PLATES FOR IMPACT WRENCH ACCESS, IN GENERAL ALL SPLICE PLATES' WIDTHS WERE CHOSEN SUCH THAT CLEAR SPACING B/W EDGE OF SPLICE PL AND FACE OF NEAREST ADJACENT STIFFENER IS ≥ THE FOLLOWING:

	BAP:	SPL. PL:	PL TO ℄ STIFF:	℄ BOLT TO STIFF. FACE
PL E	– 1000	720	140	144
	– 1333	1040	147	150
PL F	– 1333	1040	147	157
PL G	– 1200	960	120	136
	– 1600	1360	120	136
PL H	– 1175	880	148	144
	– 1325	1040	143	139
	– 1500	1280	110	126

→ SUBTRACTING SPLICE PL THICKNESS AND BOLT HEAD (SAY 35mm) ⇒ 65–70 CLEAR ON AVG.
→ SHOULD BE O.K.

CHECK NET SECTION TENSION ON SPLICES:

→ TO DETERMINE IF THE SPLICE NET SECTION IS A PROBLEM, AN ALLOWABLE MAX PLATE STRESS (IN THE PLATE BEING SPLICED) WAS COMPUTED FOR EVERY SPLICE IN A SPREADSHEET AS FOLLOWS:



CAPACITY OF NET SECTION

→ $N_{u,Rd} = \dfrac{0.9\, F_u\, A_{NET}}{\gamma_{M2}}$ F_u TAKEN AS 540 MPa. γ_{M2} TAKEN AS 1.25

→ MAX ALLOWABLE TENSION STRESS ON PLATE BEING SPLICED (FOR A GIVEN A_{NET})

$= \dfrac{N_{u,Rd}}{A_{g\,PLATE}} \ngtr F_y \Rightarrow \sigma_{T,ALLOW} = \dfrac{0.9\, F_u\, A_{NET}}{\gamma_{M2}\, A_g}$ (A_{NET} OF SPLICE PLATES; A_g OF PL. BEING SPLICED)

→ THIS WAS A MINIMUM OF 229 MPa FOR ALL SPLICE LOCATIONS.

→ MAX TENSION IN ALL TOWER PLATES AT ALL CROSS SECTIONS FOR GOVERNING LOAD COMBINATIONS CAN BE FOUND IN THE ATTACHED SPREADSHEET...

→ WORST CASE (MAX) TENSION = 193 MPa < 229 MPa ⇒ O.K. ✓

→ FOR FREESTANDING TOWER W/ TIE.BACK + EXTREME WIND, WORST CASE TENSION IN ANY PLATE IS 172 MPa $\ll 229$ MPa ⇒ O.K. ✓

→ THEORETICAL WORST-CASE NET SECTION TENSION D/C = 193/229 = 84% – O.K. ✓

∴ NET SECTION IS NOT A PROBLEM FOR THE PLATE SPLICES.

Tower Leg Vertical Steel Splices

NOTE: When skin plate is a different thickness on either side of the stiffener, the assumption for all calculations is that the stiffener is connected to the thicker skin plate

Check on Effects of Eccentricity Between Vertical Panels

		750 to 750 Stiffener 80 to 65 Skin	700 to 725 Stiffener 50 to 65 Skin	675 to 725 Stiffener 75 to 60 Skin	650 to 700 Stiffener 45 to 60 Skin	625 to 675 Stiffener 65 to 50 Skin
Larger Section						
Total Height of Larger Tower Segment	(mm)	20000	20000	20000	20000	20000
Number of Levels of Transverse Support		6	6	6	6	6
Distance Between Each Level (Typical Panel Height)		3333	3333	3333	3333	3333
Skin Plate Width - Side 1	(mm)	1000	625	625	800	625
Skin Plate Width - Side 2	(mm)	1000	625	625	800	625
Skin Plate Thickness - Side 1	(mm)	80	65	75	60	65
Skin Plate Thickness - Side 2	(mm)	80	65	75	60	65
Vertical Stiffener Width	(mm)	750	725	725	700	675
Vertical Stiffener Thickness	(mm)	75	73	73	70	68
Total Cross Section Area of Panel	(mm^2)	216250	134175	146675	145000	127150
Total Section Depth	(mm)	830	790	800	760	740
Neutral Axis from Outside (Far Side) of Skin Plate	(mm)	147.9	188.3	181.8	158.4	166.1
Neutral Axis from Extreme Fibre of Stiffener Flange	(mm)	682	602	618	602	574
I Section	(mm^4)	9.89E+09	7.35E+09	7.77E+09	6.71E+09	5.79E+09
i of Section	(mm)	213.9	234.0	230.2	215.2	213.3
e1	(mm)	307.1	239.2	255.7	251.6	236.4
e2	(mm)	107.9	155.8	144.3	128.4	133.6
e	(mm)	307.1	239.2	255.7	251.6	236.4
gamma M0		1.05	1.05	1.05	1.05	1.05
E Vertical Steel	(MPa)	210000	210000	210000	210000	210000
Fy Vertical Steel	(MPa)	460	460	460	460	460
Ncr - Euler Buckling Load	(N)	1844798597	1370524232	1450247871	1252430935	1079428827
lamda bar		0.23	0.21	0.22	0.23	0.23
alpha		0.49	0.49	0.49	0.49	0.49
alpha e		0.619	0.582	0.590	0.595	0.590
phi Factor		0.537	0.526	0.528	0.536	0.537
X (Chi Factor)		0.979	0.993	0.990	0.981	0.980
Nb,Rd	(kN)	92784	58347	63640	62319	54589
Magnification Factor (1/(1-N/Ncr))		1.053	1.044	1.046	1.052	1.053
Smaller Section						
Total Height of Larger Tower Segment	(mm)	20000	20000	20000	20000	20000
Number of Levels of Transverse Support		6	6	6	6	6
Distance Between Each Level (Typical Panel Height)		3333	3333	3333	3333	3333
Skin Plate Width - Side 1	(mm)	1000	625	625	800	625
Skin Plate Width - Side 2	(mm)	1000	625	625	800	625
Skin Plate Thickness - Side 1	(mm)	65	50	60	45	50
Skin Plate Thickness - Side 2	(mm)	65	50	60	45	50
Vertical Stiffener Width	(mm)	750	700	675	650	625
Vertical Stiffener Thickness	(mm)	75	70	68	65	63
Total Cross Section Area of Panel	(mm^2)	186250	111500	120900	114250	101875
Total Section Depth	(mm)	815	750	735	695	675
Neutral Axis from Outside (Far Side) of Skin Plate	(mm)	155.6	189.8	169.5	151.0	155.4
Neutral Axis from Extreme Fibre of Stiffener Flange	(mm)	659	560	565	544	520
I Section	(mm^4)	9.20E+09	5.88E+09	5.61E+09	4.71E+09	4.05E+09
i of Section	(mm)	222.3	229.6	215.4	203.1	199.3
e1	(mm)	284.4	210.2	228.0	219.0	207.1
e2	(mm)	123.1	164.8	139.5	128.5	130.4
e	(mm)	284.4	210.2	228.0	219.0	207.1
gamma M0		1.05	1.05	1.05	1.05	1.05
E Vertical Steel	(MPa)	210000	210000	210000	210000	210000
Fy Vertical Steel	(MPa)	460	460	460	460	460
Ncr - Euler Buckling Load	(N)	1716527440	1096140957	1046622824	879503115	754784060
lamda bar		0.22	0.22	0.23	0.24	0.25
alpha		0.49	0.49	0.49	0.49	0.49
alpha e		0.605	0.572	0.585	0.587	0.584
phi Factor		0.532	0.528	0.535	0.543	0.545
X (Chi Factor)		0.985	0.990	0.982	0.973	0.970
Nb,Rd	(kN)	80397	48374	51986	48703	43309
Magnification Factor (1/(1-N/Ncr))		1.049	1.046	1.052	1.059	1.061

Tower Leg Vertical Steel Splices

Check on Large Section at Splice Connection

NOTE: At this location, the moment is assumed to be the full axial capacity of the smaller section times the eccentricity between sections

The max compressive stress from this applied moment is compared with the "extra" available stress of the larger section with the smaller section's axial capacity applied to it as a load.

		750 to 750 Stiffener	700 to 725 Stiffener	675 to 725 Stiffener	650 to 700 Stiffener	625 to 675 Stiffener
		80 to 65 Skin	50 to 65 Skin	75 to 60 Skin	45 to 60 Skin	65 to 50 Skin
Eccentricity of Smaller Section Minus Eccentricity of Larger Section	(mm)	7.6	1.5	-12.3	-7.4	-10.6
Fy of Vertical Steel	(MPa)	460	460	460	460	460
Gamma M0		1.05	1.05	1.05	1.05	1.05
Axial Capacity of Smaller Section Including Buckling Reduction	(kN)	80397	48374	51986	48703	43309
Stress on Larger Section from Applied Axial Capacity of Smaller Section	(MPa)	372	361	354	336	341
Total Allowable Stress on Vertical Steel	(MPa)	438	438	438	438	438
Additional Stress Capacity available for Bending in Large Section at Splice	(MPa)	66	78	84	102	97
Total Moment from Axial Load and Eccentricity	(kN-m)	613	72	-640	-361	-460
S Extreme Stiffener Fibre of Larger Section	(mm^3)	14500064	12210967	12576931	11160775	10082564
S Extreme Skin Plate Fibre of Larger Section	(mm^3)	66846455	39017479	42757143	42383737	34845767
Max Bending Stress in Stiffener	(MPa)	42.3	5.9	-50.9	-32.3	-45.6
Max Bending Stress in Skin Plate	(MPa)	-9.2	-1.8	15.0	8.5	13.2
Check Ratio of Max Applied Bending Stress to Available Bending Stress	{D/C}	64%	8%	18%	8%	14%
Ceck Ratio of Total Applied Compressive Stress to Total Stress Capacity	{D/C}	95%	84%	84%	79%	81%

Check on Large Section at First Full Vertical Panel Away From Splice

NOTE: Assumes Moment distribution of point moment at end of 2 equal span continuous beam.

Max moment in this panel is same as section above, minimum moment is -1/4 max moment at opposite end

Total Height of Larger Tower Segment	(mm)	20000	20000	20000	20000	20000
Distance Between Each Level (Typical Panel Height)	(mm)	3333	3333	3333	3333	3333
Distance from Splice to First Transverse Stiffener	(mm)	2333	2333	2333	2333	2333
Relative Depth of Point to Check on First Panel (Fraction of Panel height)		0.5	0.5	0.5	0.5	0.5
Distance from Splice to Check Location on First Panel	(mm)	4000	4000	4000	4000	4000
Axial Compressive Capacity of Smaller Section	(kN)	80397	48374	51986	48703	43309
Axial Compressive Capacity of Larger Section	(kN)	92784	58347	63640	62319	54589
Interpolated Axial Demand at Check Location	(kN)	82875	50368	54317	51426	45565
Peak Moment from Eccentricity = Moment at Panel end Nearest Splice	(kN-m)	613	72	-640	-361	-460
Moment at Panel End Furthest from Splice	(kN-m)	-153.2	-18.0	160.0	90.2	115.0
Interpolated Moment Demand at Check Location	(kN-m)	229.8	27.1	-240.0	-135.3	-172.5
Max Bending Stress in Stiffener	(MPa)	15.8	2.2	-19.1	-12.1	-17.1
Max Bending Stress in Skin Plate	(MPa)	-3.4	-0.7	5.6	3.2	5.0
Buckling Capacity of Larger Section	(kN)	92784	58347	63640	62319	54589
Total Allowable Stress on Section Based on Buckling Capacity	(MPa)	429	435	434	430	429
Total Applied Stress on Section from Interpolated Axial Demand at Check Location	(MPa)	383	375	370	355	358
Available Stress for Bending due to Eccentricity	(MPa)	46	59	64	75	71
Check Ratio of Max Applied Bending Stress to Available Bending Stress	{D/C}	35%	4%	9%	4%	7%
Ceck Ratio of Total Applied Compressive Stress to Total Stress Capacity	{D/C}	93%	87%	87%	83%	85%

Check on Large Section at Second Full Vertical Panel Away From Splice

NOTE: Assumes Moment distribution of point moment at end of 2 equal span continuous beam.

Max moment in this panel is minimum moment from section above, minimum moment is 0 at opposite end

Relative Depth of Point to Check on Second Panel (Fraction of Panel height)		0.5	0.5	0.5	0.5	0.5
Distance from Splice to Check Location on Second Panel	(mm)	7333	7333	7333	7333	7333
Interpolated Axial Demand at Check Location	(kN)	84939	52031	56259	53696	47445
Peak Moment from Eccentricity = Moment at Panel end Nearest Splice	(kN-m)	-153.2	-18.0	160.0	90.2	115.0
Moment at Panel End Furthest from Splice	(kN-m)	0	0	0	0	0
Interpolated Moment Demand at Check Location	(kN-m)	-76.6	-9.0	80.0	45.1	57.5
Max Bending Stress in Stiffener	(MPa)	-5.3	-0.7	6.4	4.0	5.7
Max Bending Stress in Skin Plate	(MPa)	1.1	0.2	-1.9	-1.1	-1.7
Buckling Capacity of Larger Section	(kN)	92784	58347	63640	62319	54589
Total Allowable Stress on Section Based on Buckling Capacity	(MPa)	429	435	434	430	429
Total Applied Stress on Section from Interpolated Axial Demand at Check Location	(MPa)	393	388	384	370	373
Available Stress for Bending due to Eccentricity	(MPa)	36.3	47.1	50.3	59.5	56.2
Check Ratio of Max Applied Bending Stress to Available Bending Stress	{D/C}	3%	0%	13%	7%	10%
Ceck Ratio of Total Applied Compressive Stress to Total Stress Capacity	{D/C}	92%	89%	90%	87%	88%

Tower Leg Vertical Steel Splices

Bolted Splice Design

		750 to 750 Stiffener	700 to 725 Stiffener	675 to 725 Stiffener	650 to 700 Stiffener	625 to 675 Stiffener
Design of Slip Critical Bolted Splice on Vertical Stiffener						
Smaller Panel Vertical Stiffener Width	(mm)	750	700	675	650	625
Smaller Panel Vertical Stiffener Thickness	(mm)	75	70	68	65	63
Smaller Panel Skin Plate Thickness	(mm)	65	50	60	45	50
Total Width of Smaller Panel = Skin Thickness + Stiffener Width	(mm)	815	750	735	695	675
Larger Panel Vertical Stiffener Width	(mm)	750	725	725	700	675
Larger Panel Vertical Stiffener Thickness	(mm)	75	73	73	70	68
Larger Panel Skin Plate Thickness	(mm)	80	65	75	60	65
Total Width of Larger Panel = Skin Thickness + Stiffener Width	(mm)	830	790	800	760	740
Minimum Gap Between Edge of Splice Plate and Inside Face of Skin Plate	(mm)	95	45	100	75	50
Minimum Gap Between Edge of Splice Plate and Tip of Vertical Stiffener	(mm)	0	0	0	0	0
Maximum Available Width of Splice Plate	(mm)	640	640	560	560	560
Minimum Required Thickness of Each Splice Plate to Provide Same Area as Stiffener	(mm)	43.95	38.28	40.98	37.72	35.16
Thickness Used	(mm)	45	40	42	38	36
Thickness Check		OK	OK	OK	OK	OK
Moment of Inertia of Smaller Stiffener About Inside Face of Skin Plate	(mm^4)	1.05E+10	8.00E+09	6.97E+09	5.95E+09	5.13E+09
Moment of Inertia of Splice Plates About Inside Face of Skin Plate	(mm^4)	1.19E+10	8.57E+09	8.02E+09	6.48E+09	5.44E+09
Ratio of Supplied I Splice Plates over Required I Splice Plates		1.13	1.07	1.15	1.09	1.06
Check		OK	OK	OK	OK	OK
Bolt Spacing and Critical Distances						
Typical Bolt Diameter Used (d)	(mm)	30	30	30	30	30
Bolt Hole Diameter (d0)	(mm)	33	33	33	33	33
Minimum Bolt Edge Distance	(mm)	39.6	39.6	39.6	39.6	39.6
Edge distance Used (e2)		40	40	40	40	40
Check on Edge Distance		OK	OK	OK	OK	OK
Minimum Bolt End Distance	(mm)	39.6	39.6	39.6	39.6	39.6
End distance Used (e1)		50	50	50	50	50
Check on End Distance		OK	OK	OK	OK	OK
p1 = Minimum distance Between Transverse Lines of Bolts		72.6	72.6	72.6	72.6	72.6
p1 Used	(mm)	75	75	75	75	75
Check on p1		OK	OK	OK	OK	OK
p2 = Minimum Distance Between Lines of Bolts Parallel to Stiffener	(mm)	79.2	79.2	79.2	79.2	79.2
p2 Used	(mm)	80	80	80	80	80
Check on p2		OK	OK	OK	OK	OK
Fy of Vertical Steel	(MPa)	460	460	460	460	460
Gamma M0		1.05	1.05	1.05	1.05	1.05
Sectional Capacity of Smaller Stiffener	(kN)	24643	21467	20109	18510	17250
Check Net Section Capacity of Splice Plate						
Net Width of Splice Plate	(mm)	376	376	329	329	329
Net Area of Plate	(mm2)	33840	30080	27636	25004	23688
Fu Plate Steel	(MPa)	540	540	540	540	540
Gamma M2		1.25	1.25	1.25	1.25	1.25
Nu,Rd	(kN)	13157	11695	10745	9722	9210
Max Allowable Tensile Stress on Section Spliced (to Satisfy Net Section)	(MPa)	234	239	234	230	234
Worst Case Factored Tension Stress at All Locations in Tower Vertical Steel	(MPa)	193	193	193	193	193
Worst Possible Net Section Tension D/C at this Location		83%	81%	82%	84%	83%
Check Bearing Resistance of Bolt						
fub - Bolt Ultimate Strength	(MPa)	1000	1000	1000	1000	1000
fu - Plate Ultimate Strength	(MPa)	540	540	540	540	540
alpha d - End bolt		0.505	0.505	0.505	0.505	0.505
alpha d - Inner bolt		0.508	0.508	0.508	0.508	0.508
alpha b - End Bolt		0.505	0.505	0.505	0.505	0.505
alpha b - Inner Bolt		0.508	0.508	0.508	0.508	0.508
k1 - Edge Bolts		1.694	1.694	1.694	1.694	1.694
k1 - Inner Bolts		1.694	1.694	1.694	1.694	1.694
Gamma M2		1.25	1.25	1.25	1.25	1.25
Governing Stiffener Thickness for Bearing	(mm)	75	70	68	65	63
Fb,Rd - End and Edge Bolt (Corner Bolt)	(kN)	832	776	754	721	699
Fb,Rd - End and Inner Bolt	(kN)	832	776	754	721	699
Fb,Rd - Edge and Inner Bolt	(kN)	836	780	758	724	702
Fb,Rd - Inner and Inner Bolt	(kN)	836	780	758	724	702
Governing Bearing Resistance per Bolt	(kN)	832	776	754	721	699
Check Slip Resistance of Bolt		750 to 750 Stiffener	700 to 725 Stiffener	675 to 725 Stiffener	650 to 700 Stiffener	625 to 675 Stiffener
ks		1	1	1	1	1
n		2	2	2	2	2
slip factor (u)		0.45	0.45	0.45	0.45	0.45
Gamma M3		1.25	1.25	1.25	1.25	1.25
As Bolt (Tensile Area)	(mm^2)	562	562	562	562	562
Fp,C	(kN)	393	393	393	393	393
Fs,Rd	(kN)	283	283	283	283	283
Ratio of Slip Resistance to Bearing Resistance		0.34	0.36	0.38	0.39	0.41
Check		OK - Slip Governs	OK - Slip Governs	OK - Slip Governs	OK - Slip Governs	OK - Slip Governs
Determine the Bolt Layout for one Side of the Splice						
Number of Bolts Required for Fy Stiffener Based on Fs,Rd		88	76	71	66	61
Number of Bolts Used Per Side of Splice		88	76	73	66	61
		OK	OK	OK	OK	OK
Max Number of Bolts in a Transverse Row		8.000	8.000	7.000	7.000	7.000
Total Number of Full Transverse Bolt Rows Required		11	9	10	9	8
Remaining Number of Bolts for Last Transverse Row		0	4	3	3	5
Total Number of Transverse Bolt Rows Required		11	10	11	10	9
Length of One Side of Splice Plate	(mm)	850	775	850	775	700
Gap Between Spliced Stiffeners	(mm)	20	20	20	20	20
Total Length of Splice Plate	(mm)	1720	1570	1720	1570	1420
Total Width of Splice Plate	(mm)	640	640	560	560	560
Check Local Buckling of the Splice Plates						
epsilon		0.715	0.715	0.715	0.715	0.715
Thickness of Splice Plate	(mm)	45	40	42	38	36
p1	(mm)	75	75	75	75	75
p1/t		1.67	1.88	1.79	1.97	2.08
9*epsilon		6.43	6.43	6.43	6.43	6.43
Check that p1/t < 9*epsilon		OK - No Local Buckling	OK - No Local Buckling	OK - No Local Buckling	OK - No Local Buckling	OK - No Local Buckling
Unsupported Length of Splice Plate at Gap	(mm)	120	120	120	120	120
Lu/t		2.67	3.00	2.86	3.16	3.33
Check that Lu/t < 9*epsilon		OK - No Local Buckling	OK - No Local Buckling	OK - No Local Buckling	OK - No Local Buckling	OK - No Local Buckling

Tower Leg Vertical Steel Splices

Internal Plate and Bolt Data		
Total Width Between Vertical Stiffener Centrelines	(mm)	1500
Larger Segment Vertical Stiffener Thickness	(mm)	55
Larger Segment Plate Thickness	(mm)	50
Smaller Segment Vertical Stiffener Thickness	(mm)	55
Smaller Segment Plate Thickness	(mm)	50
Gap Between Edge of Splice Plate and Face of Vertical Stiffener	(mm)	82.5
Splice Plate Width	(mm)	1280
Splice Gap	(mm)	20
Bolt End Distance - e1	(mm)	50
Bolt Edge Distance - e2	(mm)	40
Bolt Spacing Between Transverse Rows - p1	(mm)	75
Bolt Spacing Between Longitudinal Rows Parallel to Load - p2	(mm)	80
Fy of Vertical Steel	(MPa)	460
Gamma M0		1.05
Total Axial Capacity of Plate Section Being Spliced	(kN)	32857
Capacity per Bolt Based on Slip (from other sheet)	(kN)	283.2
Minimum Number of Bolts Required to Carry Axial Load		117
Number of Bolts Used		124
Number of Bolts per Transverse Row Based on Spacings		16.00
Number of Full Transverse Rows of Bolts		7
Number of Bolts Remaining in Last Transverse Row		12.00
Total Number of Transverse Rows		8
Length of Splice Plate	(mm)	1270
Check		OK
Maximum Lever Arm of Any Force Component	(mm)	600
Magnitude of Max Force Component	(kN)	13.8
Applied Shear Stress on Section	(MPa)	19.0
Total Shear Force on Section	(kN)	1426
Centroid of Bolt Group from Centreline of Splice	(mm)	314.0
Lever Arm Between Upper and Lower Bolt Groups	(mm)	628.1
Total Applied Moment on Connection	(kN-m)	448
Total Resisting Moment Based on Max Force	(kN-m)	451
Ratio - Applied Moment / Resisting Moment		99%
Total Vertical Force on Each Bolt From Axial Demand	(kN)	265
Total Horizontal Force on Each Bolt From Shear Demand	(kN)	11.5
Worst Case Bolt D/C - Total Demand		98.6%
Size the Splice Plates for Axial and Shear Demands		
Thickness of Splice Plate Required to Provide Same Area as Plate	(mm)	29.30
Thickness of Splice Plate Used	(mm)	30.00
Axial Stress on Splice Plate From Assumed Axial Demand	(MPa)	427.8
Shear Stress on Splice Plate From Assumed Shear Demand	(MPa)	18.6
Combined Stress Using Von Mises	(MPa)	429.0
Allowable Stress = Yield / gamma M0	(MPa)	438.1
D/C Von Mises / Allowable		98%
Check the Net Section in Tension - Determine Allowable Tension Stress in Internal Plate Being Spliced		
Bolt Hole Diameter	(mm)	33
Net Width of Splice Plate	(mm)	752
Net Area of Plate	(mm2)	45120
Fu Plate Steel	(MPa)	540
Gamma M2		1.25
Nu,Rd	(kN)	17543
Max Allowable Tensile Stress on Section Spliced (to Satisfy Net Section)	(MPa)	234
Worst Case Factored Tension Stress at All Locations in Tower Vertical Steel	(MPa)	193
Worst Possible Net Section Tension D/C at this Location		83%
Summary of Splice Plate Data		
Total Number of Bolts		**248**
Plate Width (a)	(mm)	**1280**
Plate Length (b)	(mm)	**1270**
Plate Thickness (t)	(mm)	**30**
Fill Plate Thickness (t fill) - assume 1 side only	(mm)	**0**

SPLICES AT TOWER TOP → SEGMENT 21 TO SADDLE:

→ THERE ARE CURRENTLY THE FOLLOWING STIFFENER SIZES TO SPLICE.

		# BOLTS REQ'D (min)		
① → 750 x 75	→	176	→ 640 x 1720	→ 8C, 11R + 0
② → 650 x 65	→	132	→ 560 x 1570	→ 7C, 9R + 3
③ → 625 x 63	→	122	→ 560 x 1420	→ 7C, 8R + 5
④ → 600 x 60	→	112	→ 480 x 1570	→ 6C, 9R + 2
⑤ → 550 x 55	→	96	→ 480 x 1270	→ O.K. ✓

→ ① → MUST INCREASE PLATE WIDTH TO 720, REMOVE 1 ROW OF 8 BOLTS IN FAVOR OF 1 EXTRA COLUMN OF 10 BOLTS

→ 9C, 10R = 90 BOLTS TOTAL.

AT 75 SPACING, HEIGHT OF PLATE

= 2(50) + 9(75) + 10 = 785 ES. SPLICE ℄

→ LEAVES ONLY 15mm B/W BOTT OF SPLICE PL + DIAPHRAGM (IF 800 CLEAR)

→ TOO TIGHT

→ SQUEEZE OUT EVERY AVAILABLE mm:

→ END DIST'S → USE 40 (NOT 50).

STILL BARELY O.K. ✓.

→ LONGIT SPACING → USE 73 (NOT 75)

→ THIS WILL SAVE 2(10) + 9(2) = 38mm IN LENGTH ON EACH HALF

→ PROVIDES 53 CLEAR TOTAL.

→ MAY BE JUST OK...

∴ USE THE FOLLOWING ARRANGEMENTS:

- 750x75 → 180 BOLTS, 720 x 1494 PL.
- 625 x 63 → 140 BOLTS, 560 x 1494 PL.
- 600 x 60 → 126 BOLTS, 560 x 1348 PL.
- 550x55 → 96 BOLTS, 480 x 1202 PL.

UPDATED FOR NEW DEMANDS MAR 1, 2011.

→ FOR ②

CAN'T REMOVE ROW — WILL BE TOO CLOSE TO INSIDE FACE OF SKIN W/ EXTRA COLUMN.

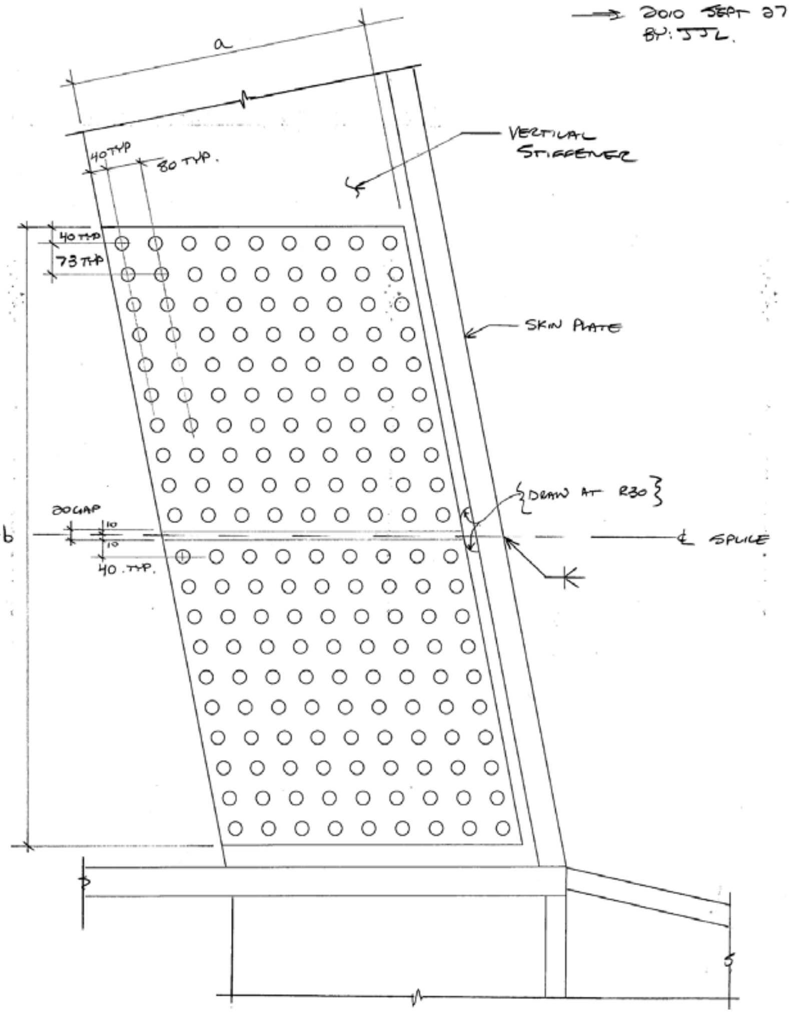
∴ 40 END DIST'S,
73 SPACING → SAME AS ①...

→ ③ → THIS IS OK...

④ → WIDEN TO 560 PL → 7C, 8R
→ TOTAL HEIGHT REDUCES TO 601 E.S. SPLICE
{2(40) + 7(73) + 10}

⑤ O.K. AS IS...

→ 2010 SEPT 27
BY: JJL.



5.2 Transverse Elements

5.2.1 Transverse Stiffeners and Type 1 Diaphragms

The following calculations are presented in this section:

- Proportioning of the transverse stiffener flanges on plate D. The flanges as proportioned allow the bracing connections shown on the general concept submission drawings between the transverse stiffener flange and longitudinal stiffeners to be removed.
- Proportioning of the triangular plate diaphragms between plates B, C, E, F and H. The finite element analysis of the triangular plate diaphragms considered a circular cut-out of 1,500 mm diameter. This cut-out was later reduced in diameter to 1,000, which is sufficient for providing the access; analysis results are therefore conservative.
- Proportioning of the welds between the plate D transverse stiffener webs and the tower leg plates. The welds as sized allow for the tab plates shown on the general concept submission drawings between the transverse and longitudinal stiffeners to be omitted.

All calculations in this section are based on the results of the detailed finite element analyses described in Section 6.

As described in CG.10.00-P-RX-D-P-SV-T4-00-00-00-00-01 "Specialist Technical Design Report, Towers," the transverse stiffeners on plate G were initially sized using the standard provisions of EN 1993-1-5, but the resulting plate thicknesses were considered too thin to be appropriate for a structure of this size. Therefore, the stiffeners were increased in size to provide more robustness. Calculations are not provided for these stiffeners.

INTRODUCTION

PURPOSE

The purpose of this folder is (1) to check the Finite Element Analysis by TNT for the stability of the flanges of the transverse stiffeners and (2) to find stiffener sizes that can be used without intermediate restraints.

CONCLUSIONS

1) For transverse stiffeners for plate D, stiffeners have flanges as follows, spanning without intermediate restraint for the clear distance between plates H:-

see page 535 (and 533)

420 × 25 mm → σ_{cr} from FE = 395 MPa → resistance $\frac{\chi f_y}{\gamma_{M1}}$ = 195 MPa
so can restrain 91 mm plate + 70×700 stiffs on 3.00 metres c/c

390 × 20 mm → σ_{cr} from FE = 358 MPa $\frac{\chi f_y}{\gamma_{M}}$ = 183 MPa
so can restrain 68 mm plate + 65×650 stiffs on 3.00 metres c/c

360 × 20 mm → σ_{cr} from FE = 329 MPa $\frac{\chi f_y}{\gamma_{M1}}$ = 173 MPa
so can restrain 60 mm plate + 65×650 stiffs on 3.00 metres c/c

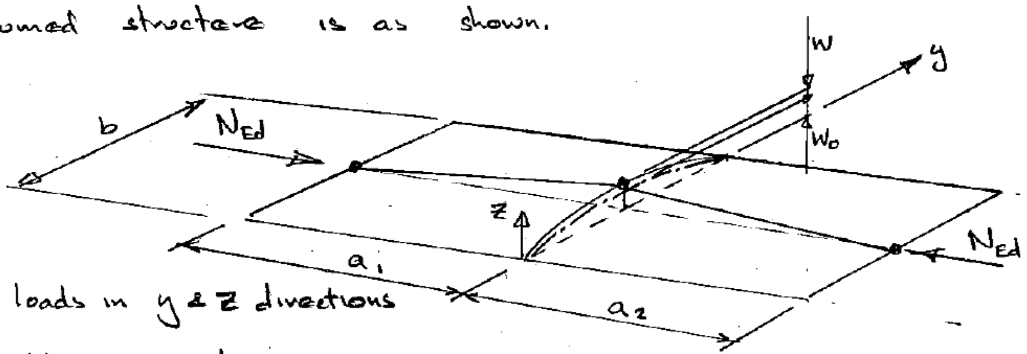
2) Approximate methods as on page 535 are safe and give sizes not far from derived by FE

3) For 100 mm plate and 70×700 stiffeners, need 450×25 flange, as page 535 for 3000 mm between stiffeners.

DEVIATION FORCES & MOMENTS

The deviation forces are calculated in accordance with EN 1993-1-5 Section 9.2. A discussion of the fundamentals of this Section can be found in the JRC Scientific & Technical Report "Commentary and Worked Examples to EN1993-1-5 "Plated Structural Elements" by Johansson, Maquoi, Sedlacek, Müller and Beg, published by ECCS, October 2007.

The assumed structure is as shown.



Assuming no loads in y & z directions

At the equilibrium position:

Destabilising load/unit width $= \left(\frac{w+w_o}{a_1} + \frac{w+w_o}{a_2}\right)\frac{N_{Ed}}{b} \sin\frac{\pi y}{b}$

Stabilizing load/unit width $= \frac{d^2M}{dy^2} = \frac{d^2}{dy^2}\left(EI\frac{d^2z}{dy^2}\right) = EI\,w\frac{\pi^4}{b^4}\sin\frac{\pi y}{b}$

Stabilising = Destabilizing

$$\therefore EI\,w\frac{\pi^4}{b^4}\left(\sin\frac{\pi y}{b}\right) = \left(\frac{w+w_o}{a_1} + \frac{w+w_o}{a_2}\right)\frac{N_{Ed}}{b}\left(\sin\frac{\pi y}{b}\right)$$

$$\therefore EI\left(\frac{\pi}{b}\right)^4 w = \left(\frac{1}{a_1}+\frac{1}{a_2}\right)\frac{N_{Ed}}{b}(w+w_o)$$

$$\therefore EI\left(\frac{\pi}{b}\right)^4 w - \left(\frac{1}{a_1}+\frac{1}{a_2}\right)\left(\frac{N_{Ed}}{b}\right)w = \left(\frac{1}{a_1}+\frac{1}{a_2}\right)\left(\frac{N_{Ed}}{b}\right)w_o$$

$$\therefore \left[EI\left(\frac{\pi}{b}\right)^4 - \left(\frac{1}{a_1}+\frac{1}{a_2}\right)\left(\frac{N_{Ed}}{b}\right)\right]w = \left(\frac{1}{a_1}+\frac{1}{a_2}\right)\left(\frac{N_{Ed}}{b}\right)w_o$$

$$\therefore w = \frac{\left(\frac{1}{a_1}+\frac{1}{a_2}\right)\frac{N_{Ed}}{b}}{\left[EI\left(\frac{\pi}{b}\right)^4 - \left(\frac{1}{a_1}+\frac{1}{a_2}\right)\frac{N_{Ed}}{b}\right]}\,w_o$$

Bending moment, $M = EI\frac{d^2z}{dy^2} = EI\frac{d^2}{dx^2}\left(w\sin\frac{\pi y}{b}\right) = EIw\left(\frac{\pi}{b}\right)^2\sin\frac{\pi y}{b} \Rightarrow M_{max} = EIw\left(\frac{\pi}{b}\right)^2$

Distributed load, $q = \frac{d^2M}{dy^2} = \frac{d^2}{dy^2}\left(EIw\left(\frac{\pi}{b}\right)^2\sin\frac{\pi y}{b}\right) = EIw\left(\frac{\pi}{b}\right)^4\sin\frac{\pi y}{b} \Rightarrow q_{max} = EIw\left(\frac{\pi}{b}\right)^4$

DEVIATION FORCES & MOMENTS (cont)

For Plate D with $a_1 = a_2 = 3000$ mm
$b = 8000$ mm

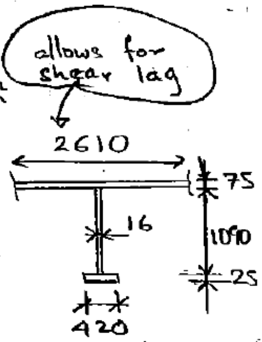
5 No. longitudinal stiffeners @ 68 × 675 mm → $A = 5 \times 68 \times 675 = 229500\ mm^2$
$t = 75$ mm → $A = 75 \times 8000 = 600000$
$829500\ mm^2$

Max longitudinal design force $N_{Ed} = \dfrac{A f_y}{\gamma_{M0}} = 829500 \times \dfrac{460}{1.05} = 363.4$ MN

#9.2.1(2) $w_0 = \dfrac{s}{300}$ where s is the minimum of a_1, a_2, b
$a_1 = 3000$
$a_2 = 3000$
$b = 8000$

$\therefore w_0 = \dfrac{3000}{300} = 10$ mm

$$\left(\frac{1}{a_1} + \frac{1}{a_2}\right)\frac{N_{Ed}}{b} = \left(\frac{1}{3000} + \frac{1}{3000}\right)\frac{363.4 \times 10^6}{8000} = 30.28\ N/mm^2$$



I of transverse stiffener $= 19.19 \times 10^9\ mm^4$

$$\therefore EI\left(\frac{\pi}{b}\right)^4 = 210 \times 10^3 \times 19.19 \times 10^9 \left(\frac{\pi}{8000}\right)^4 = 95.84 \times 10^9\ N/mm^2$$

$$\therefore w = \frac{30.28}{[95.84 - 30.28]} w_1 = 0.462 \times 10 = 4.62\ mm$$

$\therefore$ Max bending moment $= EIw\left(\dfrac{\pi}{b}\right)^2$

$$= 210 \times 10^3 \times 19.19 \times 10^9 \times 4.62 \times \left(\frac{\pi}{8000}\right)^2 = 2871 \times 10^6\ N\text{-}mm = 2871\ kN\text{-}m$$

Max distributed load $= EIw\left(\dfrac{\pi}{b}\right)^4$

$$= 210 \times 10^3 \times 19.19 \times 10^9 \times 4.62 \times \left(\frac{\pi}{8000}\right)^4 = 443\ N/mm = 443\ kN/m$$

Modulus	Es	210,000	210,000	210,000	210,000	210,000
Poisson's ratio	nu	0.30	0.30	0.30	0.30	0.30
Shear modulus	G	80,769	80,769	80,769	80,769	80,769
f_y		460	460	460	460	460
GammaM0	GammaM0	1.05	1.05	1.05	1.05	1.05
GammaM1	GammaM	1.1	1.1	1.1	1.1	1.1
fy/GammaM1		418	418	418	418	418
axial stress	s_a	0	0	0	0	0
available bending	s_bmax	418	418	418	418	418
Segment		**1**	**1**	**1**	**2**	**2**
Tower		**Sicilia**	**Sicilia**	**Calabria**	**Sicilia**	**Calabria**
transverse stiffener span	b	8000	8000	8000	8000	8000
transverse stiffener spacing	a_1	**3000**	**3000**	**3000**	**3000**	**3000**
transverse stiffener spacing	a_2	**3000**	**3000**	**3000**	**3000**	**3000**
plate CL eccy 1	e_1	0	0	0	0	0
plate CL eccy 2	e_2	0	0	0	0	0
number of longitudinal stiffeners	nls	**5**	**5**	**5**	**5**	**5**
thickness of longitudinal stiffeners	tls	**65**	**65**	**68**	**65**	**65**
breadth of longitudinal stiffeners	bls	**650**	**650**	**675**	**650**	**650**
skin breadth	bskf	2610	2610	2610	2610	2610
skin thickness	tskf	**70**	**70**	**75**	**55**	**60**
transverse stiffener web breadth	bstw	1090	1090	1090	1090	1090
transverse stiffener web thickness	tstw	16.0	16.0	16.0	16.0	16.0
transverse stiffener flange breadth	bstf	**420**	**390**	**420**	**420**	**420**
transverse stiffener flange thickness	tstf	**25**	**20**	**25**	**25**	**25**
transverse stiffener depth	Dst	1185	1180	1190	1170	1175
Area	A_s	210640	207940	223690	171490	184540
Skin; distance of centroid from outside face of skin	yski	35	35	37.5	27.5	30
Web; distance of centroid from outside face of skin	ywi	615	615	620	600	605
Flange; distance of centroid from outside face of skin	yfi	1172.5	1170	1177.5	1157.5	1162.5
Neutral axis; distance of centroid from outside face of skin	yNA	139.72	126.22	136.43	154.91	148.78
Skin; distance of centroid from NA	yNAski	104.7	91.2	98.9	127.4	118.8
Web; distance of centroid from NA	yNAwi	-475.3	-488.8	-483.6	-445.1	-456.2
Flange; distance of centroid from NA	yNAfi	-1032.8	-1043.8	-1041.1	-1002.6	-1013.7
I provided	I	18,944,596,725	1.6E+10	19,193,193,092	18,103,143,669	18,403,653,160
centroid of skin from NA	ec_sk	104.72	91.22	98.93	127.41	118.78
centroid of stiffener from NA	ec_st	684.79	660.29	693.09	654.60	665.73
e centroid	e_c	685	660	693	655	666
y0 = e_max	y	1045.3	1053.8	1053.6	1015.1	1026.2
radius of gyration	is	299.9	277.3	292.9	324.9	315.8
alpha_s	apha_s	0.49	0.49	0.49	0.49	0.49
alpha_e	alpha_e	0.696	0.704	0.703	0.671	0.680
lambda_s = b/is		26.7	28.9	27.3	24.6	25.3
lambda1_s =		67.1	67.1	67.1	67.1	67.1
lambdab_s		0.397	0.430	0.407	0.367	0.377
phi_s=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)		0.648	0.673	0.655	0.623	0.632
chi_s =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1		0.863	0.839	0.855	0.887	0.879
resistance stress =chi_s*f_y/GammaM		360.8	350.9	357.6	371.0	367.5
axial compressive force applied to tranvserse stiffener	N	0	0	0	0	0
Ncr = PI()^2*Es*I/b^2	Ncr	613,513,934	5.2E+08	621,564,638	586,263,780	595,995,672
((f_y/GammaM)-(N/A_s))		418.2	418.2	418.2	418.2	418.2
(1-(N/Ncr))		1.000	1.000	1.000	1.000	1.000
M=sb*I/y		7,579,129,172	6.3E+09	7,618,113,061	7,457,860,295	7,499,421,595
w_0c*m=M/N		-1000.00	-1000.00	-1000.00	-1000.00	-1000.00
w_0c1=(fy/gM-N/A)(I/(N*y))(1-N/Ncr)	w_0c1	-1000.00	-1000.00	-1000.00	-1000.00	-1000.00
w_0c2 = b/300	w_0c2	26.67	26.67	26.67	26.67	26.67
w_0c=MAX(w_0c1,w_0c2)	w_0c	26.67	26.67	26.67	26.67	26.67
e_max	e_max	1045.3	1053.8	1053.6	1015.1	1026.2
w_0=MIN(a_1/300, a_2/300, b/300)	w_0	10.00	10.00	10.00	10.00	10.00
Horizontal force/m	NL_per_m	0	0	0	0	0
Longitudinal force	NL	337,880,952	3.4E+08	363,400,000	285,309,524	302,833,333
Area of plate + stiffener	A_ps	0	0	0	0	0
External compression force applied to transverse stiffener	Na	0	0	0	0	0
Es*I*(PI()/b)^4	f_4	94.6	79.8	95.9	90.4	91.9
Q = (NL/b)*(1/a_1+1/a_2)+Na*(PI()/b)^2	Q	28.2	28.2	30.3	23.8	25.2
Fd	Fd	0	0	0	0	0
w_1 = (w_0*Q+Fd)/(f_4-Q)	w_1	4.24	5.45	4.62	3.57	3.79
Max distributed load on stiffener = Es*I*w_1*(PI()/b)^4		401	435	443	323	348
Max bending moment in stiffener = Es*I*w_1*(PI()/b)^2	M	2,599,443,902	2.8E+09	2,870,691,169	2,091,875,280	2,255,846,770
max bending stress = M*y/I	sbEd	143.4	185.9	157.6	117.3	125.8

Stiffener lateral buckling								
Inertia of stiffener flange + half web	Istz		154,350,000	9.9E+07	154,350,000		154,350,000	154,350,000
Area of stiffener flange + half web	Astz		19,220.00	1.7E+04	19,220.00		19,220.00	19,220.00
radius of gyration	isz		90	77	90		90	90
alpha_sz	apha_sz		0.49	0.49	0.49		0.49	0.49
alpha_ez	alpha_ez		0.595	0.603	0.595		0.595	0.595
Unrestrained length	**Lcrz**		**8000**	**8000**	**8000**		**8000**	**8000**
lambda_sz = Lcr/isz = b/isz (because Lcr = b)			89.3	103.4	89.3		89.3	89.3
lambda1_sz =	lambda1_sz		67.1	67.1	67.1		67.1	67.1
It	It		3,675,713	2.5E+06	3,675,713		3,675,713	3,675,713
Distance of flange centroid from face of skin	yf		1102.5	1100.0	1102.5		1102.5	1102.5
Polar moment of area of flange+web about face of skin	Ip		19,823,986,958	1.6E+10	19,823,986,958		19,823,986,958	19,823,986,958
modified for moment Ip	IpM	0	18,097,281,625	1.5E+10	18,097,281,625		18,097,281,625	18,097,281,625
G*It			296,884,538,462	2.0E+11	296,884,538,462		296,884,538,462	296,884,538,462
E*Istz*yf^2*(PI()/Lcrz)^2			6,075,791,979,347	3.9E+12	6,075,791,979,347		6,075,791,979,347	6,075,791,979,347
(E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			306	236	306		306	306
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			321	248	321		321	321
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/IpM	s_crzIpM	#	352	277	352		352	352
s_crz = (PI()^2*Es*Istz/b^2)/Astz	s_crz		260	194	260		260	260
FE s_cr	**s_crzFE**		**395**	**358**	**395**		**329**	**329**
elastic critical buckling stress (MUST SELECT)	s_cr	#	352	358	395		329	329
lambdab_sz from Lcr/isz			1.330	1.541	1.330		1.330	1.330
lambdab_sz from s_cr	lambdab_sz		1.143	1.134	1.079		1.182	1.182
phi_sz=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)	phi_sz		1.434	1.424	1.344		1.492	1.492
chi_sz =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1	chi_sz		0.435	0.437	0.466		0.417	0.417
resistance stress =chi_sz*f_y/GammaM	s_bRdz		181.8	182.9	194.9		174.2	174.2
D/C = sbEd/s_bRdz			**0.789**	**1.016**	**0.808**		**0.673**	**0.722**
				Not used				

Modulus	Es	210,000	210,000	210,000	210,000
Poisson's ratio	nu	0.30	0.30	0.30	0.30
Shear modulus	G	80,769	80,769	80,769	80,769
f_y		460	460	460	460
GammaM0	GammaM0	1.05	1.05	1.05	1.05
GammaM1	GammaM	1.1	1.1	1.1	1.1
fy/GammaM1		418	418	418	418
axial stress	s_a	0	0	0	0
available bending	s_bmax	418	418	418	418
Segment		**3**	**7**	**8**	**8**
Tower		**Sicilia**	**Sicilia**	**Sicilia**	**Sicilia**
transverse stiffener span	b	8000	8000	8000	8000
transverse stiffener spacing	a_1	**3000**	**3166**	**3166**	**3166**
transverse stiffener spacing	a_2	**3000**	**3000**	**3166**	**3166**
plate CL eccy 1	e_1	0	0	0	0
plate CL eccy 2	e_2	0	0	0	0
number of longitudinal stiffeners	nls	**5**	**5**	**5**	**5**
thickness of longitudinal stiffeners	tls	**63**	**65**	**68**	**68**
breadth of longitudinal stiffeners	bls	**625**	**650**	**675**	**675**
skin breadth	bskf	2610	2610	2610	2610
skin thickness	tskf	**50**	**60**	**70**	**70**
transverse stiffener web breadth	bstw	1090	1090	1090	1090
transverse stiffener web thickness	tstw	16.0	16.0	16.0	16.0
transverse stiffener flange breadth	bstf	**390**	**420**	**420**	**390**
transverse stiffener flange thickness	tstf	**20**	**25**	**25**	**20**
transverse stiffener depth	Dst	1160	1175	1185	1180
Area	A_s	155740	184540	210640	207940
Skin; distance of centroid from outside face of skin	yski	25	30	35	35
Web; distance of centroid from outside face of skin	ywi	595	605	615	615
Flange; distance of centroid from outside face of skin	yfi	1150	1162.5	1172.5	1170
Neutral axis; distance of centroid from outside face of skin	yNA	145.17	148.78	139.72	126.22
Skin; distance of centroid from NA	yNAski	120.2	118.8	104.7	91.2
Web; distance of centroid from NA	yNAwi	-449.8	-456.2	-475.3	-488.8
Flange; distance of centroid from NA	yNAfi	-1004.8	-1013.7	-1032.8	-1043.8
I provided	I	15,043,143,152	18,403,653,160	18,944,596,725	1.6E+10
centroid of skin from NA	ec_sk	120.17	118.78	104.72	91.22
centroid of stiffener from NA	ec_st	621.34	665.73	684.79	660.29
e centroid	e_c	621	666	685	660
y0 = e_max	y	1014.8	1026.2	1045.3	1053.8
radius of gyration	is	310.8	315.8	299.9	277.3
alpha_s	apha_s	0.49	0.49	0.49	0.49
alpha_e	alpha_e	0.670	0.680	0.696	0.704
lambda_s = b/is		25.7	25.3	26.7	28.9
lambda1_s =		67.1	67.1	67.1	67.1
lambdab_s		0.383	0.377	0.397	0.430
phi_s=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)		0.635	0.632	0.648	0.673
chi_s =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1		0.876	0.879	0.863	0.839
resistance stress =chi_s*f_y/GammaM		366.5	367.5	360.8	350.9
axial compressive force applied to tranvserse stiffener	N	0	0	0	0
Ncr = PI()^2*Es*I/b^2	Ncr	487,166,767	595,995,672	613,513,934	5.2E+08
((f_y/GammaM)-(N/A_s))		418.2	418.2	418.2	418.2
(1-(N/Ncr))		1.000	1.000	1.000	1.000
M=sb*I/y		6,198,860,715	7,499,421,595	7,579,129,172	6.3E+09
w_0c*m=M/N		-1000.00	-1000.00	-1000.00	-1000.00
w_0c1=(fy/gM-N/A)(I/(N*y))(1-N/Ncr)	w_0c1	-1000.00	-1000.00	-1000.00	-1000.00
w_0c2 = b/300	w_0c2	26.67	26.67	26.67	26.67
w_0c=MAX(w_0c1,w_0c2)	w_0c	26.67	26.67	26.67	26.67
e_max	e_max	1014.8	1026.2	1045.3	1053.8
w_0=MIN(a_1/300, a_2/300, b/300)	w_0	10.00	10.00	10.55	10.55
Horizontal force/m	NL_per_m	0	0	0	0
Longitudinal force	NL	261,488,095	302,833,333	345,876,190	3.5E+08
Area of plate + stiffener	A_ps	0	0	0	0
External compression force applied to transverse stiffener	Na	0	0	0	0
Es*I*(PI()/b)^4	f_4	75.1	91.9	94.6	79.8
Q = (NL/b)*(1/a_1+1/a_2)+Na*(PI()/b)^2	Q	21.8	24.6	27.3	27.3
Fd	Fd	0	0	0	0
w_1 = (w_0*Q+Fd)/(f_4-Q)	w_1	4.09	3.65	4.28	5.49
Max distributed load on stiffener = Es*I*w_1*(PI()/b)^4		307	335	405	438
Max bending moment in stiffener = Es*I*w_1*(PI()/b)^2	M	1,990,321,781	2,175,124,095	2,627,544,689	2.8E+09
max bending stress = M*y/I	sbEd	134.3	121.3	145.0	187.3

Stiffener lateral buckling						
Inertia of stiffener flange + half web	Istz		98,865,000	154,350,000	154,350,000	9.9E+07
Area of stiffener flange + half web	Astz		16,520.00	19,220.00	19,220.00	16,520.00
radius of gyration	isz		77	90	90	77
alpha_sz	apha_sz		0.49	0.49	0.49	0.49
alpha_ez	alpha_ez		0.603	0.595	0.595	0.603
Unrestrained length	**Lcrz**		**8000**	**8000**	**8000**	**8000**
lambda_sz = Lcr/isz = b/isz (because Lcr = b)			103.4	89.3	89.3	103.4
lambda1_sz =	lambda1_sz		67.1	67.1	67.1	67.1
It	It		2,528,213	3,675,713	3,675,713	2,528,213
Distance of flange centroid from face of skin	yf		1100.0	1102.5	1102.5	1100.0
Polar moment of area of flange+web about face of skin	Ip		16,443,686,333	19,823,986,958	19,823,986,958	1.6E+10
modified for moment Ip	IpM	0	14,716,981,000	18,097,281,625	18,097,281,625	1.5E+10
G*It			204,201,846,154	296,884,538,462	296,884,538,462	2.0E+11
E*Istz*yf^2*(PI()/Lcrz)^2			3,874,065,927,794	6,075,791,979,347	6,075,791,979,347	3.9E+12
(E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			236	306	306	236
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			248	321	321	248
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/IpM	s_crzIpM	#	277	352	352	277
s_crz = (PI()^2*Es*Istz/b^2)/Astz	s_crz		194	260	260	194
FE s_cr	**s_crzFE**		**329**	**329**	**395**	**358**
elastic critical buckling stress (MUST SELECT)	s_cr	#	329	329	395	358
lambdab_sz from Lcr/isz			1.541	1.330	1.330	1.541
lambdab_sz from s_cr	lambdab_sz		1.182	1.182	1.079	1.134
phi_sz=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)	phi_sz		1.496	1.492	1.344	1.424
chi_sz =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1	chi_sz		0.415	0.417	0.466	0.437
resistance stress =chi_sz*f_y/GammaM	s_bRdz		173.4	174.2	194.9	182.9
D/C = sbEd/s_bRdz			**0.774**	**0.696**	**0.744**	**1.024**
						Not used

Modulus	Es	210,000	210,000	210,000	210,000	210,000
Poisson's ratio	nu	0.30	0.30	0.30	0.30	0.30
Shear modulus	G	80,769	80,769	80,769	80,769	80,769
f_y		460	460	460	460	460
GammaM0	GammaM0	1.05	1.05	1.05	1.05	1.05
GammaM1	GammaM	1.1	1.1	1.1	1.1	1.1
fy/GammaM1		418	418	418	418	418
axial stress	s_a	0	0	0	0	0
available bending	s_bmax	418	418	418	418	418
Segment		**9**	**9**	**10**	**13**	**13**
Tower		**Sicilia**	**Sicilia**	**Sicilia**	**Sicilia**	**Sicilia**
transverse stiffener span	b	8000	8000	8000	8000	8000
transverse stiffener spacing	a_1	**3166**	**3166**	**3333**	**3333**	**3887**
transverse stiffener spacing	a_2	**3333**	**3333**	**3333**	**3333**	**2535**
plate CL eccy 1	e_1	0	0	0	0	0
plate CL eccy 2	e_2	0	0	0	0	0
number of longitudinal stiffeners	nls	**5**	**5**	**5**	**5**	**5**
thickness of longitudinal stiffeners	tls	**68**	**68**	**65**	**68**	**68**
breadth of longitudinal stiffeners	bls	**675**	**675**	**650**	**675**	**675**
skin breadth	bskf	2610	2610	2610	2610	2610
skin thickness	tskf	**65**	**65**	**50**	**55**	**55**
transverse stiffener web breadth	bstw	1090	1090	1090	1090	1090
transverse stiffener web thickness	tstw	16.0	16.0	16.0	16.0	16.0
transverse stiffener flange breadth	bstf	**420**	**390**	**390**	**390**	**390**
transverse stiffener flange thickness	tstf	**25**	**20**	**20**	**20**	**20**
transverse stiffener depth	Dst	1180	1175	1160	1165	1165
Area	A_s	197590	194890	155740	168790	168790
Skin; distance of centroid from outside face of skin	yski	32.5	32.5	25	27.5	27.5
Web; distance of centroid from outside face of skin	ywi	610	610	595	600	600
Flange; distance of centroid from outside face of skin	yfi	1167.5	1165	1150	1155	1155
Neutral axis; distance of centroid from outside face of skin	yNA	143.79	129.50	145.17	138.76	138.76
Skin; distance of centroid from NA	yNAski	111.3	97.0	120.2	111.3	111.3
Web; distance of centroid from NA	yNAwi	-466.2	-480.5	-449.8	-461.2	-461.2
Flange; distance of centroid from NA	yNAfi	-1023.7	-1035.5	-1004.8	-1016.2	-1016.2
I provided	I	18,682,604,617	1.6E+10	15,043,143,152	15,305,754,374	15,305,754,374
centroid of skin from NA	ec_sk	111.29	97.00	120.17	111.26	111.26
centroid of stiffener from NA	ec_st	675.72	652.01	621.34	632.76	632.76
e centroid	e_c	676	652	621	633	633
y0 = e_max	y	1036.2	1045.5	1014.8	1026.2	1026.2
radius of gyration	is	307.5	284.5	310.8	301.1	301.1
alpha_s	apha_s	0.49	0.49	0.49	0.49	0.49
alpha_e	alpha_e	0.688	0.696	0.670	0.679	0.679
lambda_s = b/is		26.0	28.1	25.7	26.6	26.6
lambda1_s =		67.1	67.1	67.1	67.1	67.1
lambdab_s		0.388	0.419	0.383	0.396	0.396
phi_s=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)		0.640	0.664	0.635	0.645	0.645
chi_s =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1		0.871	0.848	0.876	0.867	0.867
resistance stress =chi_s*f_y/GammaM		364.1	354.7	366.5	362.4	362.4
axial compressive force applied to tranvserse stiffener	N	0	0	0	0	0
Ncr = PI()^2*Es*I/b^2	Ncr	605,029,414	5.1E+08	487,166,767	495,671,337	495,671,337
((f_y/GammaM)-(N/A_s))		418.2	418.2	418.2	418.2	418.2
(1-(N/Ncr))		1.000	1.000	1.000	1.000	1.000
M=sb*I/y		7,539,687,125	6.3E+09	6,198,860,715	6,236,906,802	6,236,906,802
w_0c*m=M/N		-1000.00	-1000.00	-1000.00	-1000.00	-1000.00
w_0c1=(fy/gM-N/A)(I/(N*y))(1-N/Ncr)	w_0c1	-1000.00	-1000.00	-1000.00	-1000.00	-1000.00
w_0c2 = b/300	w_0c2	26.67	26.67	26.67	26.67	26.67
w_0c=MAX(w_0c1,w_0c2)	w_0c	26.67	26.67	26.67	26.67	26.67
e_max	e_max	1036.2	1045.5	1014.8	1026.2	1026.2
w_0=MIN(a_1/300, a_2/300, b/300)	w_0	10.55	10.55	11.11	11.11	8.45
Horizontal force/m	NL_per_m	0	0	0	0	0
Longitudinal force	NL	328,352,381	3.3E+08	267,785,714	293,304,762	293,304,762
Area of plate + stiffener	A_ps	0	0	0	0	0
External compression force applied to transverse stiffener	Na	0	0	0	0	0
Es*I*(PI()/b)^4	f_4	93.3	78.8	75.1	76.4	76.4
Q = (NL/b)*(1/a_1+1/a_2)+Na*(PI()/b)^2	Q	25.3	25.3	20.1	22.0	23.9
Fd	Fd	0	0	0	0	0
w_1 = (w_0*Q+Fd)/(f_4-Q)	w_1	3.92	4.99	4.05	4.49	3.84
Max distributed load on stiffener = Es*I*w_1*(PI()/b)^4		366	393	305	343	294
Max bending moment in stiffener = Es*I*w_1*(PI()/b)^2	M	2,372,739,654	2.5E+09	1,975,127,425	2,225,481,478	1,904,742,720
max bending stress = M*y/I	sbEd	131.6	168.8	133.2	149.2	127.7

Stiffener lateral buckling							
Inertia of stiffener flange + half web	Istz		154,350,000	9.9E+07	98,865,000	98,865,000	98,865,000
Area of stiffener flange + half web	Astz		19,220.00	16,520.00	16,520.00	16,520.00	16,520.00
radius of gyration	isz		90	77	77	77	77
alpha_sz	apha_sz		0.49	0.49	0.49	0.49	0.49
alpha_ez	alpha_ez		0.595	0.603	0.603	0.603	0.603
Unrestrained length	**Lcrz**		**8000**	**8000**	**8000**	**8000**	**8000**
lambda_sz = Lcr/isz = b/isz (because Lcr = b)			89.3	103.4	103.4	103.4	103.4
lambda1_sz =	lambda1_sz		67.1	67.1	67.1	67.1	67.1
It	It		3,675,713	2,528,213	2,528,213	2,528,213	2,528,213
Distance of flange centroid from face of skin	yf		1102.5	1100.0	1100.0	1100.0	1100.0
Polar moment of area of flange+web about face of skin	Ip		19,823,986,958	1.6E+10	16,443,686,333	16,443,686,333	16,443,686,333
modified for moment Ip	IpM	0	18,097,281,625	1.5E+10	14,716,981,000	14,716,981,000	14,716,981,000
G*It			296,884,538,462	2.0E+11	204,201,846,154	204,201,846,154	204,201,846,154
E*Istz*yf^2*(PI()/Lcrz)^2			6,075,791,979,347	3.9E+12	3,874,065,927,794	3,874,065,927,794	3,874,065,927,794
(E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			306	236	236	236	236
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			321	248	248	248	248
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/IpM	s_crzIpM	#	352	277	277	277	277
s_crz = (PI()^2*Es*Istz/b^2)/Astz	s_crz		260	194	194	194	194
FE s_cr	**s_crzFE**		**358**	**329**	**329**	**329**	**329**
elastic critical buckling stress (MUST SELECT)	s_cr	#	358	329	329	329	329
lambdab_sz from Lcr/isz			1.330	1.541	1.541	1.541	1.541
lambdab_sz from s_cr	lambdab_sz		1.134	1.182	1.182	1.182	1.182
phi_sz=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)	phi_sz		1.420	1.496	1.496	1.496	1.496
chi_sz =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1	chi_sz		0.439	0.415	0.415	0.415	0.415
resistance stress =chi_sz*f_y/GammaM	s_bRdz		183.7	173.4	173.4	173.4	173.4
D/C = sbEd/s_bRdz			**0.716**	**0.974**	**0.768**	**0.860**	**0.736**
				Not used			

Modulus	Es	210,000	210,000	210,000	210,000	210,000
Poisson's ratio	nu	0.30	0.30	0.30	0.30	0.30
Shear modulus	G	80,769	80,769	80,769	80,769	80,769
f_y		460	460	460	460	460
GammaM0	GammaM0	1.05	1.05	1.05	1.05	1.05
GammaM1	GammaM	1.1	1.1	1.1	1.1	1.1
fy/GammaM1		418	418	418	418	418
axial stress	s_a	0	0	0	0	0
available bending	s_bmax	418	418	418	418	418
Segment		**19**	**19**	**20**	**20**	**21**
Tower		**Calabria**	**Calabria**	**Sicilia**	**Sicilia**	**Sicilia**
transverse stiffener span	b	8000	8000	8000	8000	8000
transverse stiffener spacing	a_1	**3280**	**3280**	**3125**	**3125**	**2894**
transverse stiffener spacing	a_2	**3000**	**3280**	**3125**	**2894**	**2894**
plate CL eccy 1	e_1	0	0	0	0	0
plate CL eccy 2	e_2	0	0	0	0	0
number of longitudinal stiffeners	nls	**5**	**5**	**5**	**5**	**5**
thickness of longitudinal stiffeners	tls	**63**	**63**	**63**	**63**	**60**
breadth of longitudinal stiffeners	bls	**625**	**625**	**625**	**625**	**600**
skin breadth	bskf	2610	2610	2610	2610	2610
skin thickness	tskf	**55**	**55**	**60**	**60**	**60**
transverse stiffener web breadth	bstw	1090	1090	1090	1090	1090
transverse stiffener web thickness	tstw	16.0	16.0	16.0	16.0	16.0
transverse stiffener flange breadth	bstf	**390**	**390**	**420**	**420**	**420**
transverse stiffener flange thickness	tstf	**20**	**20**	**25**	**25**	**25**
transverse stiffener depth	Dst	1165	1165	1175	1175	1175
Area	A_s	168790	168790	184540	184540	184540
Skin; distance of centroid from outside face of skin	yski	27.5	27.5	30	30	30
Web; distance of centroid from outside face of skin	ywi	600	600	605	605	605
Flange; distance of centroid from outside face of skin	yfi	1155	1155	1162.5	1162.5	1162.5
Neutral axis; distance of centroid from outside face of skin	yNA	138.76	138.76	148.78	148.78	148.78
Skin; distance of centroid from NA	yNAski	111.3	111.3	118.8	118.8	118.8
Web; distance of centroid from NA	yNAwi	-461.2	-461.2	-456.2	-456.2	-456.2
Flange; distance of centroid from NA	yNAfi	-1016.2	-1016.2	-1013.7	-1013.7	-1013.7
I provided	I	15,305,754,374	15,305,754,374	18,403,653,160	18,403,653,160	18,403,653,160
centroid of skin from NA	ec_sk	111.26	111.26	118.78	118.78	118.78
centroid of stiffener from NA	ec_st	632.76	632.76	665.73	665.73	665.73
e centroid	e_c	633	633	666	666	666
y0 = e_max	y	1026.2	1026.2	1026.2	1026.2	1026.2
radius of gyration	is	301.1	301.1	315.8	315.8	315.8
alpha_s	apha_s	0.49	0.49	0.49	0.49	0.49
alpha_e	alpha_e	0.679	0.679	0.680	0.680	0.680
lambda_s = b/is		26.6	26.6	25.3	25.3	25.3
lambda1_s =		67.1	67.1	67.1	67.1	67.1
lambdab_s		0.396	0.396	0.377	0.377	0.377
phi_s=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)		0.645	0.645	0.632	0.632	0.632
chi_s =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1		0.867	0.867	0.879	0.879	0.879
resistance stress =chi_s*f_y/GammaM		362.4	362.4	367.5	367.5	367.5
axial compressive force applied to tranvserse stiffener	N	0	0	0	0	0
Ncr = PI()^2*Es*I/b^2	Ncr	495,671,337	495,671,337	595,995,672	595,995,672	595,995,672
((f_y/GammaM)-(N/A_s))		418.2	418.2	418.2	418.2	418.2
(1-(N/Ncr))		1.000	1.000	1.000	1.000	1.000
M=sb*I/y		6,236,906,802	6,236,906,802	7,499,421,595	7,499,421,595	7,499,421,595
w_0c*m=M/N		-1000.00	-1000.00	-1000.00	-1000.00	-1000.00
w_0c1=(fy/gM-N/A)(I/(N*y))(1-N/Ncr)	w_0c1	-1000.00	-1000.00	-1000.00	-1000.00	-1000.00
w_0c2 = b/300	w_0c2	26.67	26.67	26.67	26.67	26.67
w_0c=MAX(w_0c1,w_0c2)	w_0c	26.67	26.67	26.67	26.67	26.67
e_max	e_max	1026.2	1026.2	1026.2	1026.2	1026.2
w_0=MIN(a_1/300, a_2/300, b/300)	w_0	10.00	10.93	10.42	9.65	9.65
Horizontal force/m	NL_per_m	0	0	0	0	0
Longitudinal force	NL	279,011,905	279,011,905	296,535,714	296,535,714	289,142,857
Area of plate + stiffener	A_ps	0	0	0	0	0
External compression force applied to transverse stiffener	Na	0	0	0	0	0
Es*I*(PI()/b)^4	f_4	76.4	76.4	91.9	91.9	91.9
Q = (NL/b)*(1/a_1+1/a_2)+Na*(PI()/b)^2	Q	22.3	21.3	23.7	24.7	25.0
Fd	Fd	0	0	0	0	0
w_1 = (w_0*Q+Fd)/(f_4-Q)	w_1	4.11	4.21	3.62	3.54	3.60
Max distributed load on stiffener = Es*I*w_1*(PI()/b)^4		314	322	333	325	331
Max bending moment in stiffener = Es*I*w_1*(PI()/b)^2	M	2,036,341,757	2,088,872,114	2,159,912,825	2,109,371,196	2,145,551,720
max bending stress = M*y/I	sbEd	136.5	140.1	120.4	117.6	119.6

Stiffener lateral buckling							
Inertia of stiffener flange + half web	Istz		98,865,000	98,865,000	154,350,000	154,350,000	154,350,000
Area of stiffener flange + half web	Astz		16,520.00	16,520.00	19,220.00	19,220.00	19,220.00
radius of gyration	isz		77	77	90	90	90
alpha_sz	apha_sz		0.49	0.49	0.49	0.49	0.49
alpha_ez	alpha_ez		0.603	0.603	0.595	0.595	0.595
Unrestrained length	**Lcrz**		**8000**	**8000**	**8000**	**8000**	**8000**
lambda_sz = Lcr/isz = b/isz (because Lcr = b)			103.4	103.4	89.3	89.3	89.3
lambda1_sz =	lambda1_sz		67.1	67.1	67.1	67.1	67.1
It	It		2,528,213	2,528,213	3,675,713	3,675,713	3,675,713
Distance of flange centroid from face of skin	yf		1100.0	1100.0	1102.5	1102.5	1102.5
Polar moment of area of flange+web about face of skin	Ip		16,443,686,333	16,443,686,333	19,823,986,958	19,823,986,958	19,823,986,958
modified for moment Ip	IpM	0	14,716,981,000	14,716,981,000	18,097,281,625	18,097,281,625	18,097,281,625
G*It			204,201,846,154	204,201,846,154	296,884,538,462	296,884,538,462	296,884,538,462
E*Istz*yf^2*(PI()/Lcrz)^2			3,874,065,927,794	3,874,065,927,794	6,075,791,979,347	6,075,791,979,347	6,075,791,979,347
(E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			236	236	306	306	306
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/Ip			248	248	321	321	321
(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/IpM	s_crzIpM	#	277	277	352	352	352
s_crz = (PI()^2*Es*Istz/b^2)/Astz	s_crz		194	194	260	260	260
FE s_cr	**s_crzFE**		**329**	**329**	**329**	**329**	**329**
elastic critical buckling stress (MUST SELECT)	s_cr	#	329	329	329	329	329
lambdab_sz from Lcr/isz			1.541	1.541	1.330	1.330	1.330
lambdab_sz from s_cr	lambdab_sz		1.182	1.182	1.182	1.182	1.182
phi_sz=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)	phi_sz		1.496	1.496	1.492	1.492	1.492
chi_sz =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1	chi_sz		0.415	0.415	0.417	0.417	0.417
resistance stress =chi_sz*f_y/GammaM	s_bRdz		173.4	173.4	174.2	174.2	174.2
D/C = sbEd/s_bRdz			**0.787**	**0.808**	**0.691**	**0.675**	**0.687**

INTRODUCTION

PURPOSE

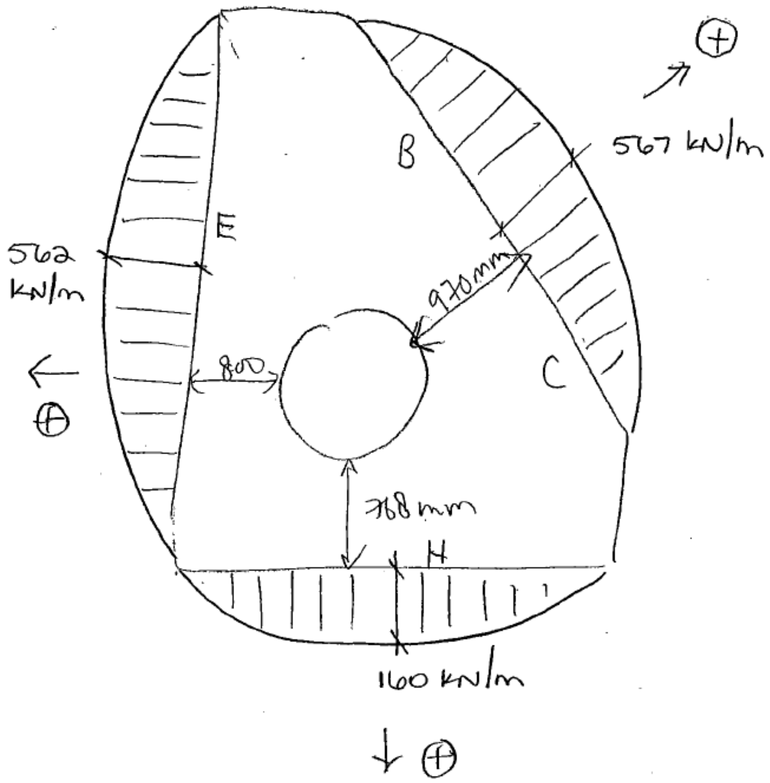
To calculate the resistance of the triangular diaphragms in the legs & the kick loads from the longitudinal forces.

CONCLUSION

20pl is OK especially with ring stiffener - page 115

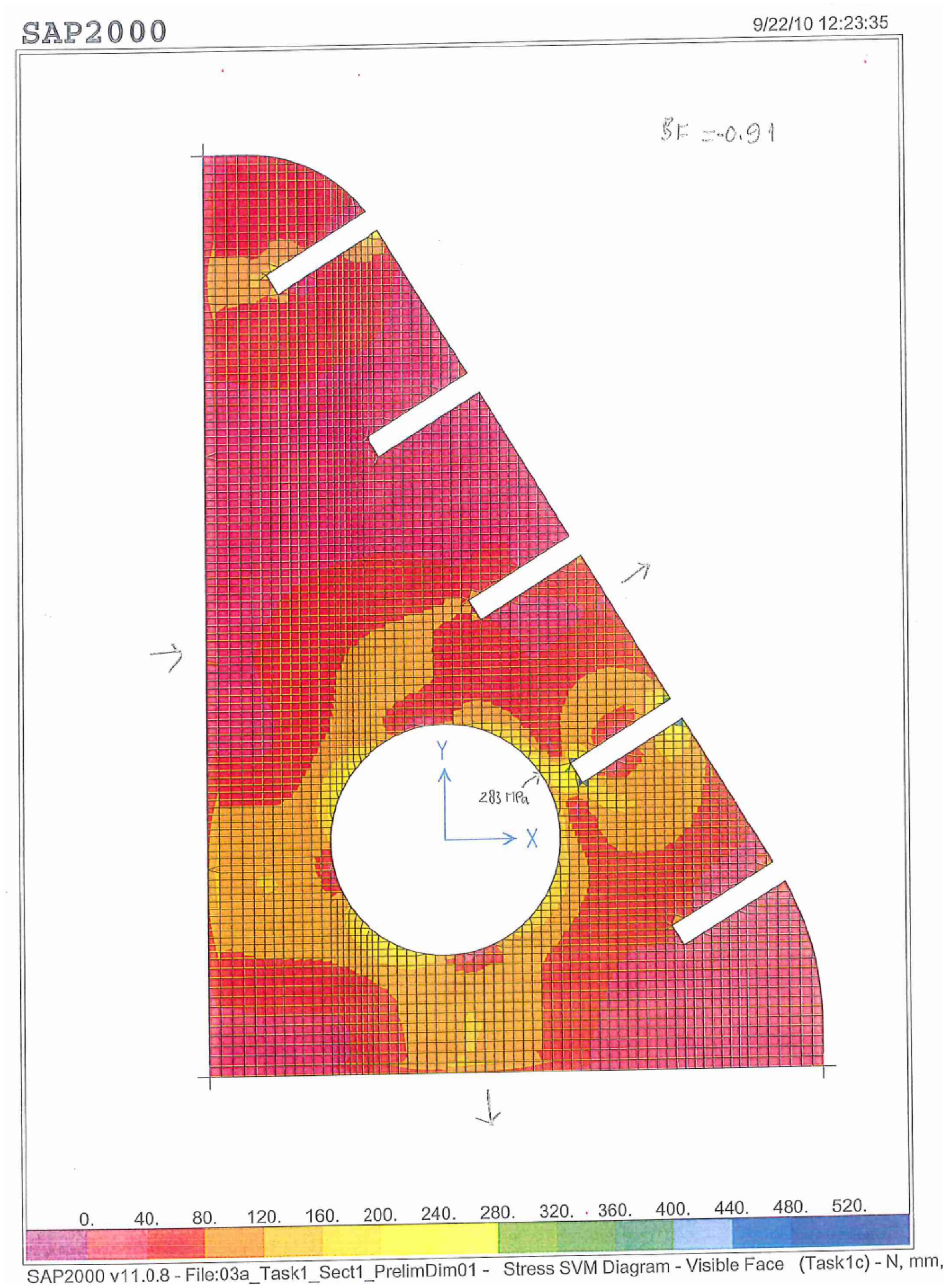
LOADING

Model 3a Loading.



		BC	H	E	BC	H	E	
Task	a	1	1	1	-1	-1	-1	e
	b	1	-1	1	-1	1	-1	f
	c	1	1	-1	-1	-1	1	g
	d	1	-1	-1	-1	1	1	h

$$2^{3-1} = 2^2 = 4$$



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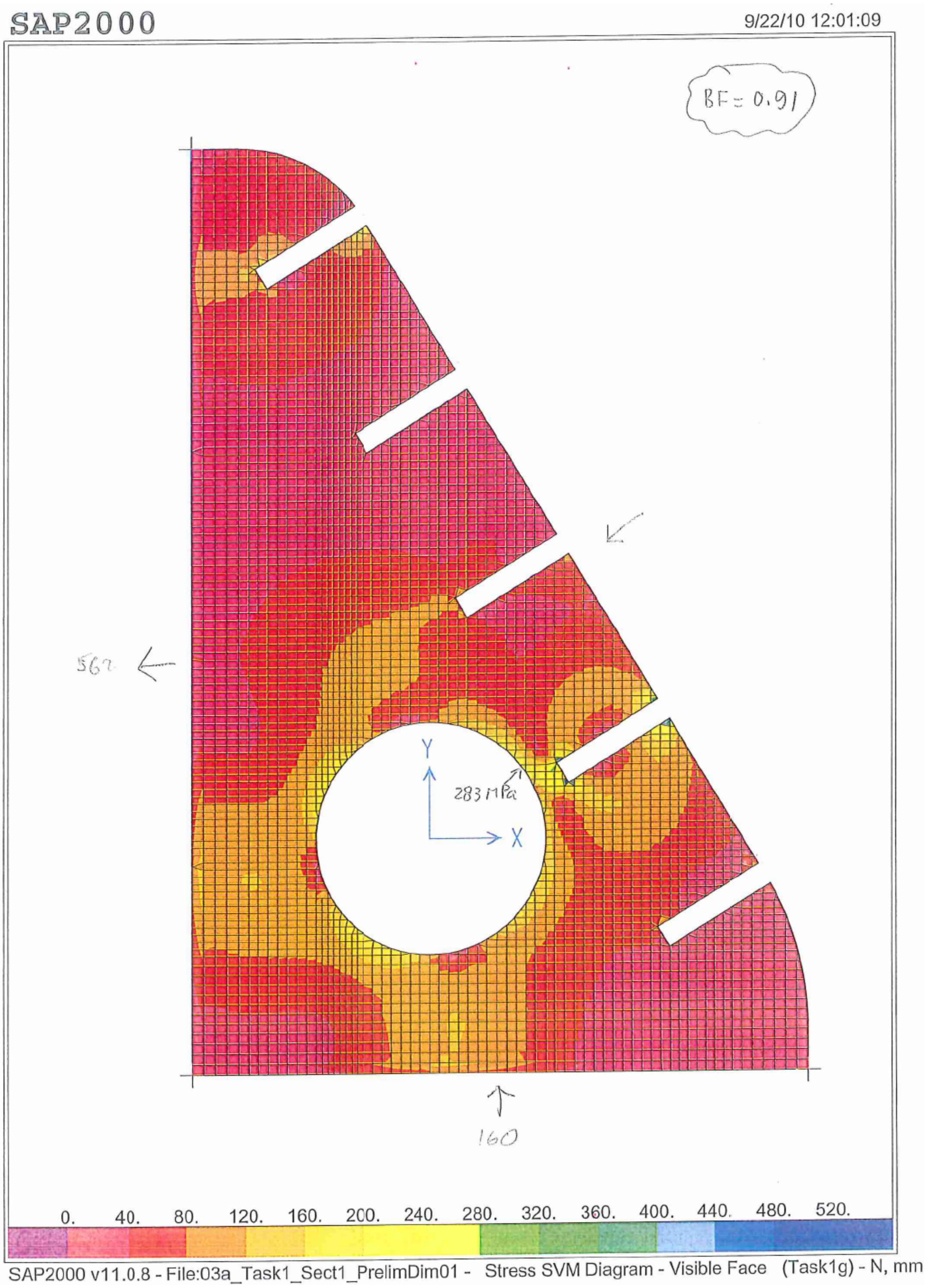
SF = 0.91
283 MPa
Y
X
0. 40. 80. 120. 160. 200. 240. 280. 320. 360. 400. 440. 480. 520.
SAP2000 v11.0.8 - File:03a_Task1_Sect1_PrelimDim01 - Stress SVM Diagram - Visible Face (Task1c) - N, mm,



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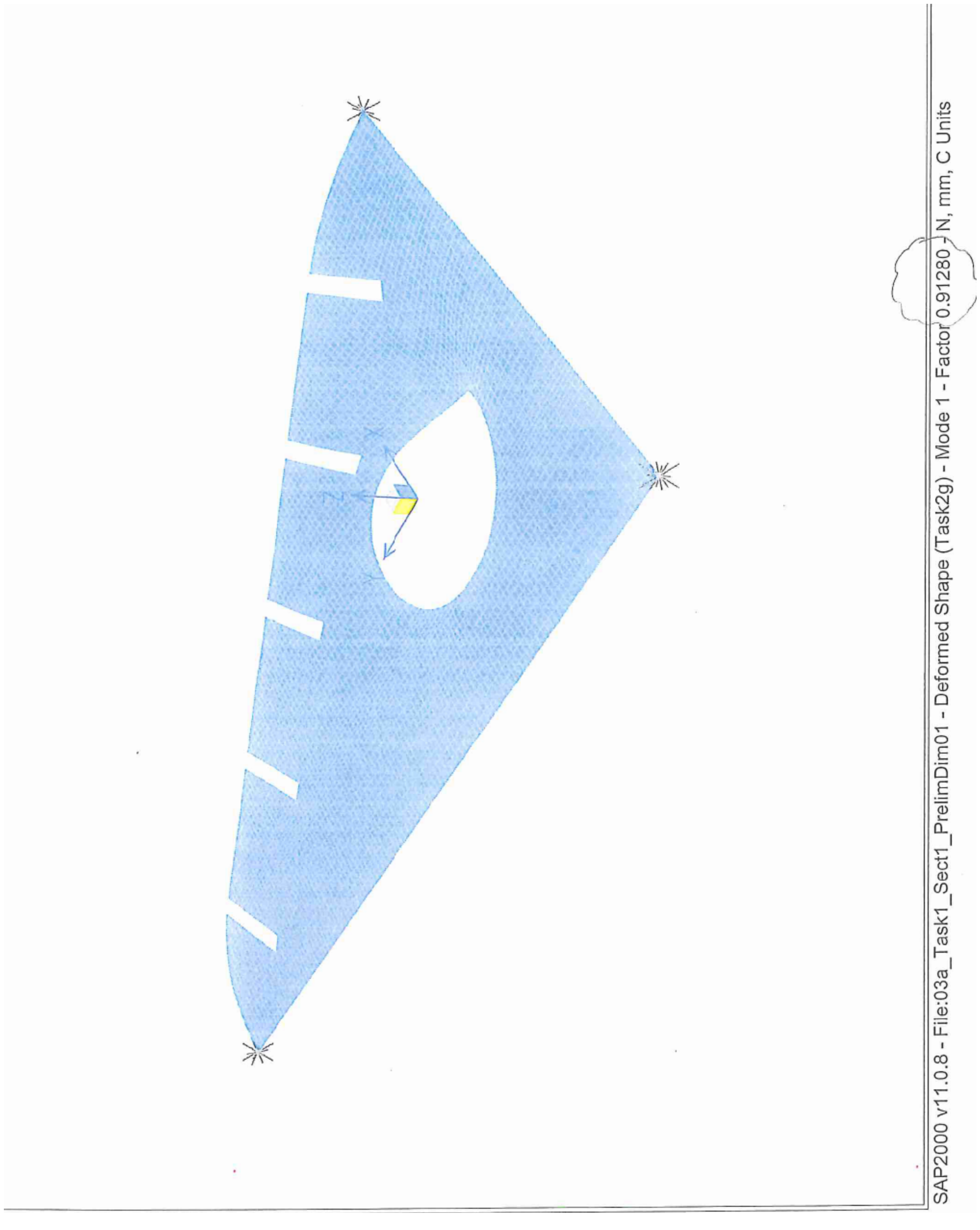
283 MPa
Y
X
160
0. 40. 80. 120. 160. 200. 240. 280. 320. 360. 400. 440. 480. 520.
SAP2000 v11.0.8 - File:03a_Task1_Sect1_PrelimDim01 - Stress SVM Diagram - Visible Face (Task1g) - N, mm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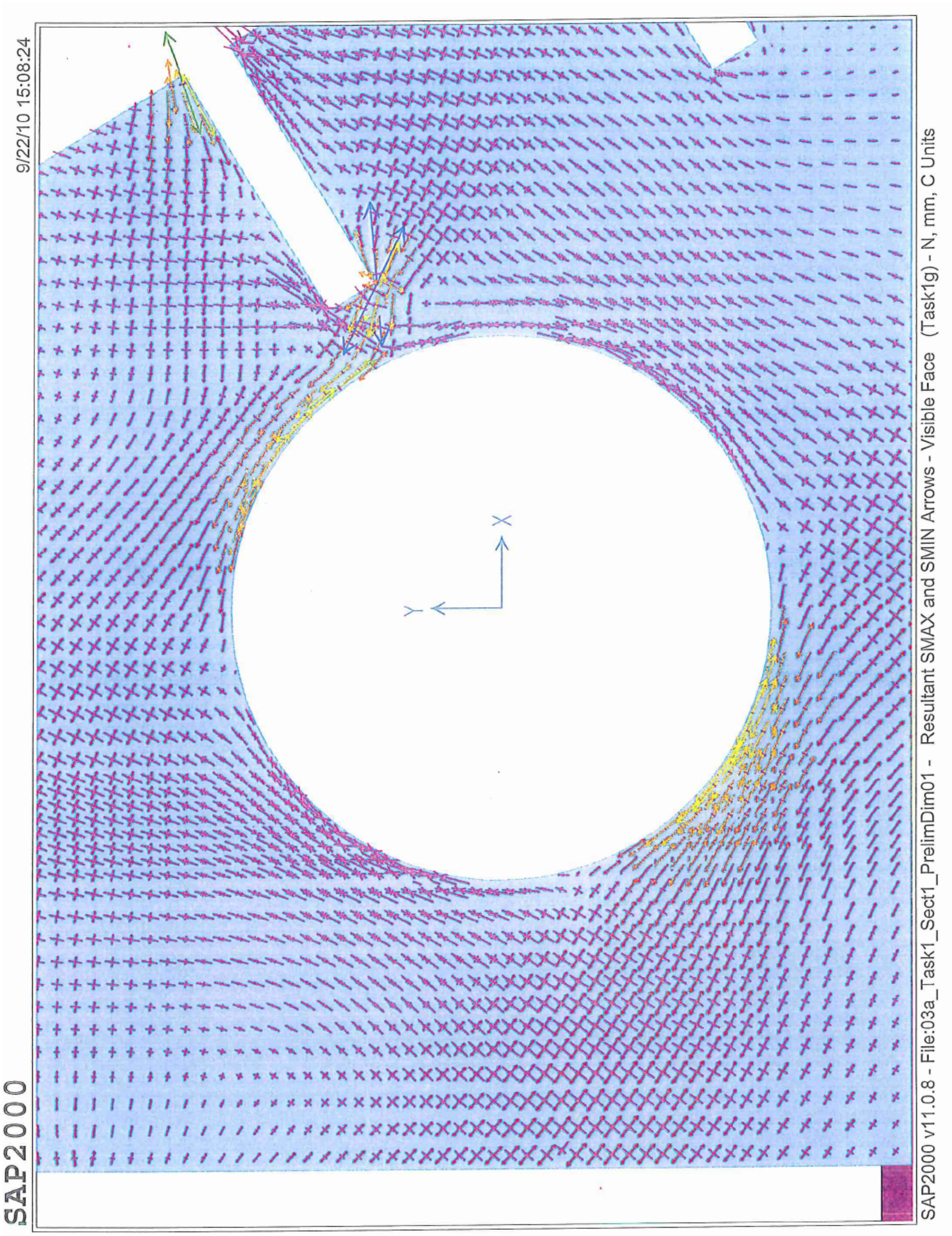

9/22/10 15:08:24
SAP2000
SAP2000 v11.0.8 - File:03a_Task1_Sect1_PrelimDim01 - Resultant SMAX and SMIN Arrows - Visible Face (Task1g) - N, mm, C Units

RESISTANCE

REVIEW LOADS

BUCKLING FACTOR α_{cr}

Member is a diaphragm, so there will be no heat induced curvature as in a transverse stiffener

Expected straightness ≈ 1/1000, but assume 1/500

NOTE: There is no appreciable deflection under load

$$\text{Lateral load} = \left(\frac{w_0}{a_1} + \frac{w_0}{a_2}\right) \times \frac{N_L}{b} \; ; \text{ where } N_L = 4\times1250\times100\times\frac{460}{1.05} = 219\text{ MN}$$

$$+\; 5\times700\times70\times\frac{460}{1.05} = \underline{107}$$

$$326\text{ MN}$$

$$\Delta t_{w_0}\!: a/500 = \left(\frac{1}{500}+\frac{1}{500}\right)\times\frac{326\times10^6}{4\times1250} = 261\text{ N/mm}$$

$$\Delta t_{w_0} = a/300 = \left(\frac{1}{300}+\frac{1}{300}\right)\times\frac{326\times10^6}{4\times1250} = 435\text{ N/mm} = 435\text{ kN/m}$$

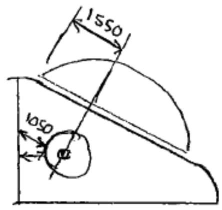
compared with maximum applied load of 567 kN/m in analysis

$$\therefore \text{Buckling factor from analysis is increased by } \frac{567}{435} = 1.303$$

$$\therefore \text{Minimum buckling factor } \alpha_{cr} = 1.303\times0.9128 = 1.189$$

(Pages 1112 & 1106)

ULTIMATE FACTOR α_{ULT}



$$1^{st}\text{ Approximation} = \frac{\text{Squash load/mm}}{\text{Max. line load}}$$

$$= \frac{18\times460}{435} = 18.6$$

$$2^{nd}\text{ Approximation} = \frac{1050\times18\times460}{1550\times435} = 12.9$$

Take $\alpha_{ult} = 6$ for conservatism

PLATE SLENDERNESS

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = \sqrt{\frac{6}{1.189}} = 2.25$$

RESISTANCE (cont)

$\phi_p = \frac{1}{2}\left(1 + 0.34(2.25 - 0.70) + 2.25\right) = 1.8575$

$\therefore \rho = \frac{1}{1.8575 + \sqrt{1.8575^2 - 2.25}} = 0.34$

$\therefore \#10(2) \rightarrow \frac{\rho \alpha_{ult k}}{\gamma_{M1}} = \frac{0.34 \times 6}{1.1} = 1.85 > 1 \Rightarrow OK$

With reference to buckling stress, $\frac{D}{C} = \frac{1}{\alpha_{cr}} = \frac{1}{1.189} = 0.84 \Rightarrow OK.$

FROM VON MISES ← (very conservative because max von Mises stress occurs locally at the edge of the hole and not at the buckling)

$\alpha_{ult} = \frac{460}{283} = 1.625$

$\therefore \bar{\lambda}_p = \sqrt{\frac{1.625}{1.189}} = 1.169$

$\therefore \phi = \frac{1}{2}\left(1 + 0.34(1.169 - 0.70) + 1.169\right) = 1.164$

$\rho = \frac{1}{1.164 + \sqrt{1.164^2 - 1.169}} = 0.627$

$\therefore \#10(2) \rightarrow \frac{\rho \alpha_{ult}}{\gamma_{M1}} = \frac{0.627 \times 1.625}{1.1} = 0.926 < 1 = \frac{C}{D}$

Model used 18 mm plate → α_{cr} increase by $\left(\frac{20}{18}\right)^2$; → α_{ult} increases by $\left(\frac{20}{18}\right)$

Drawings show 20 mm plate → resistance increases by $> \frac{20}{18} = \frac{1.0}{0.9}$

$\Rightarrow D/C > \frac{1}{0.926 \times \frac{18}{20}} = 0.972 \Rightarrow OK$

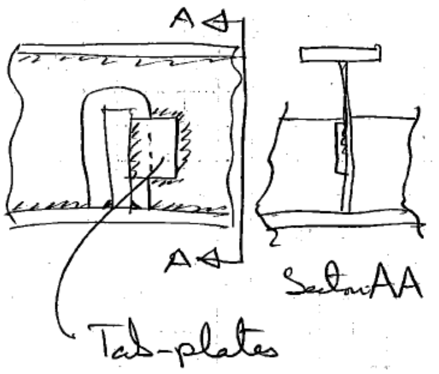
In addition, drawings show ring stiffeners.

$\therefore$ Drawings OK

INTRODUCTION

PURPOSE

The purpose of this book is to prove that the tab-plates can be removed.



CONCLUSION

Pages 473-476 show $D/C < 1.0$ for 2No fillet welds of 5mm throat

Welds on drawings are 5mm throat ⇒ Tab-plates not needed.

CG1000 P DX DP SV T4 T0 00 D0 00 01 A

ANALYSIS WITH STABLE FLANGE SIZES

From page 425, Sach & Sach $d = 6mm$
Length of FE element $\approx 35mm$
EN 1993-1-8 does not show an identical connection, but #4.10, Figure 4.8, shows load from a plate dispersing into another. Equation 4.6a uses an equivalent distribution length of $7t$ for 2-sided distribution.

Inputs from page 533 for loads/mm
420x25   479N/mm on 85 pt

390x20   315N/mm on 55 pt

360x20   270N/mm on 45 pt

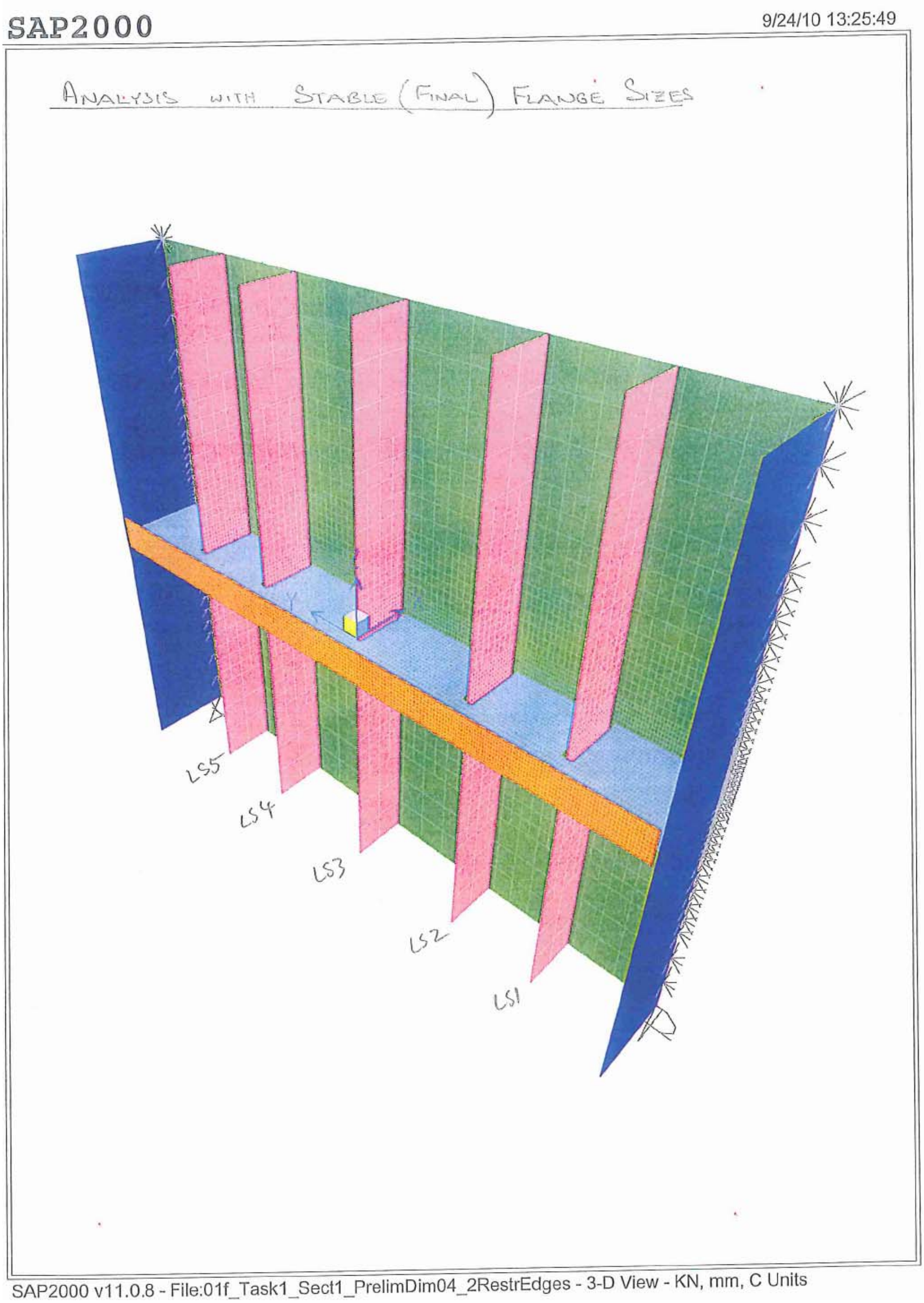
Using Figure 4.8, taking Fig 4.8 flange = Messina skin plate
Fig 4.8 plate = Messina stiffener web

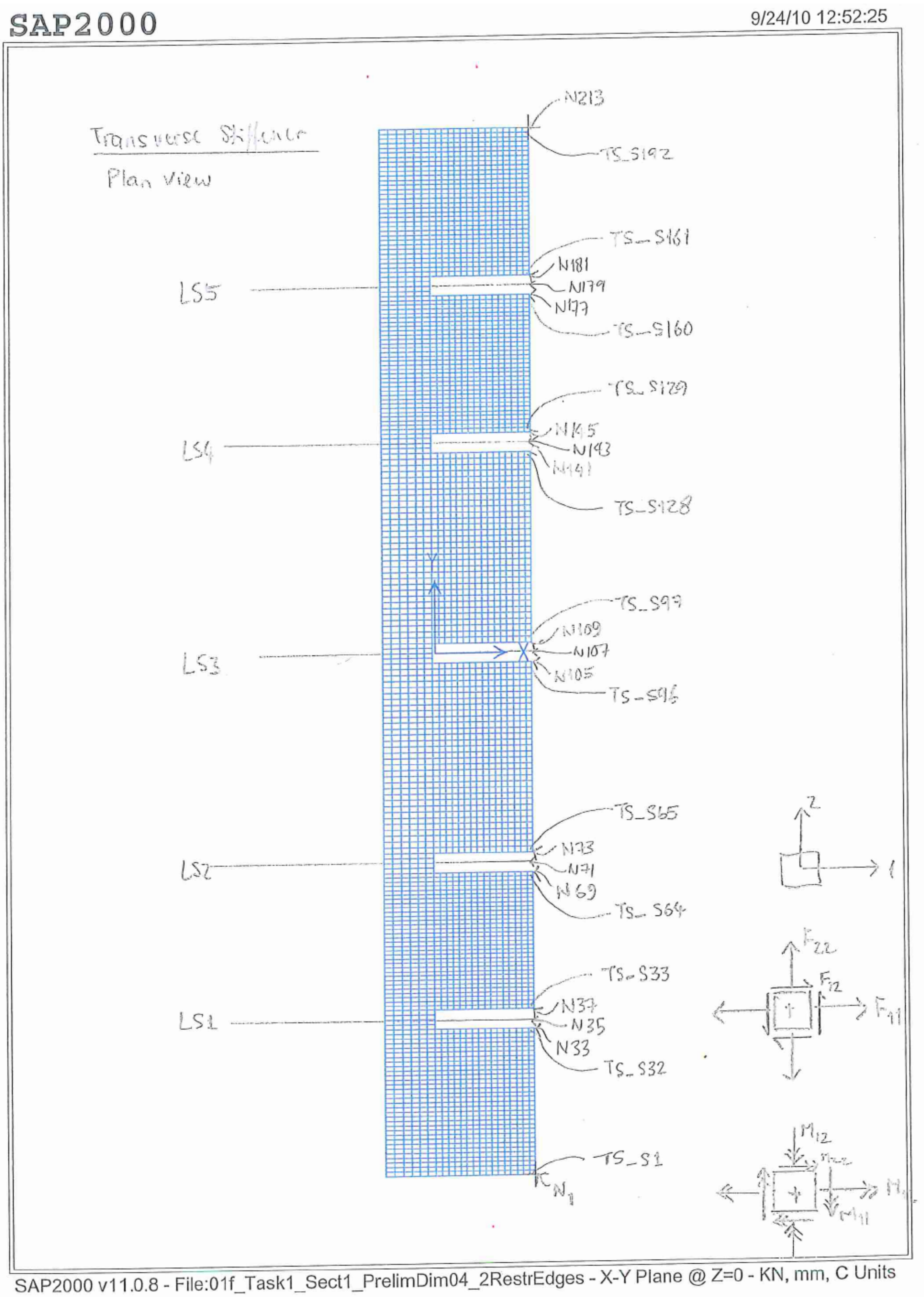
$\Rightarrow$ Equation 4.6b; $k = \left(\frac{t_f}{t_p}\right)\left(\frac{f_{yf}}{f_{yp}}\right) \approx \frac{40}{16} \times \frac{460}{460} = 2.5, \Rightarrow k = 1$
because $k \leq 1.0$
(minimum thickness)

In equation 4.6a, $t_w = 0$ because there is no plate acting as this load path
take $s = 0$ for simplicity and conservatism

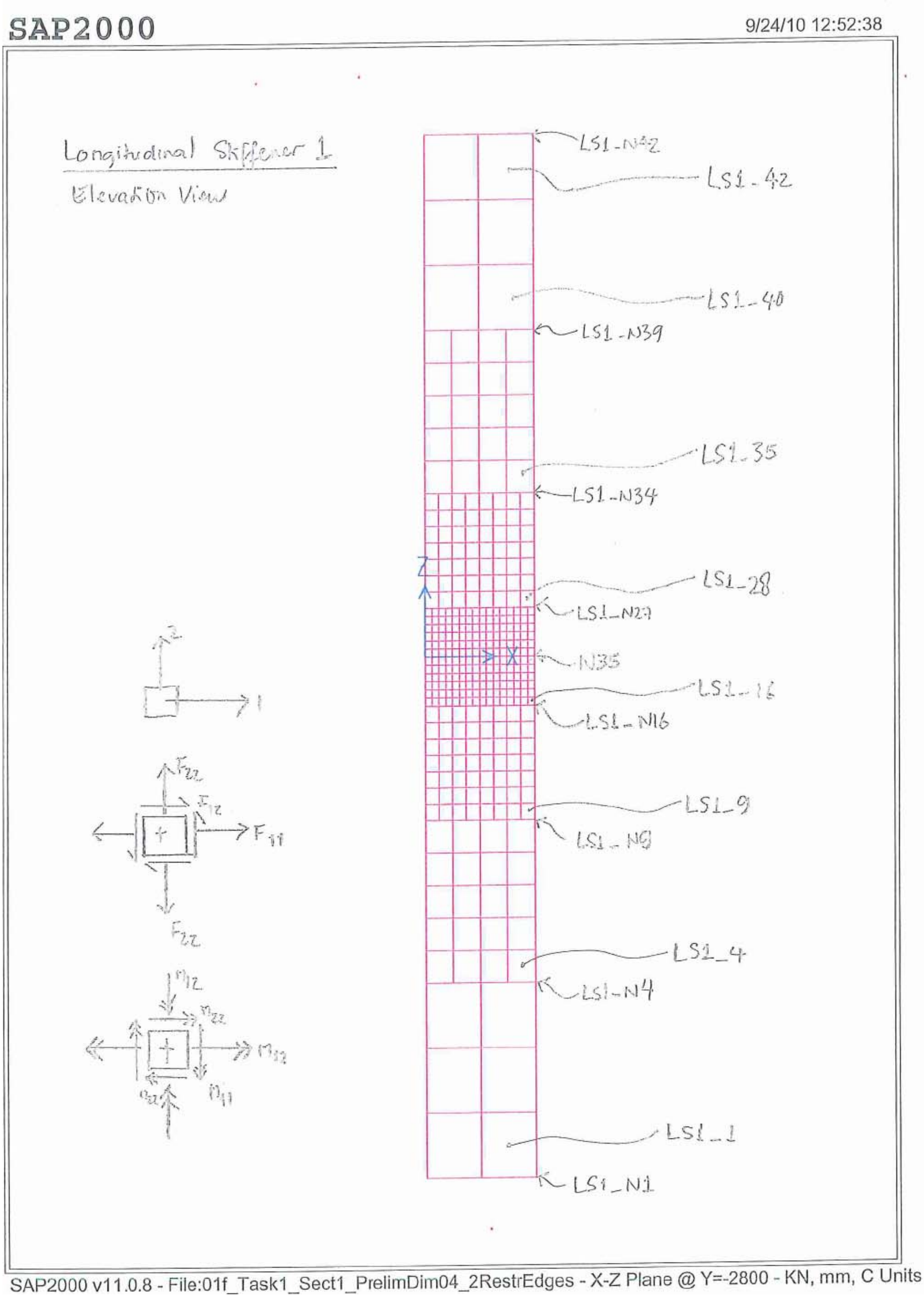
. For one sided distribution, $b_{eff} = \frac{1}{2} \times 7kt_f \approx 3.5t_f$

In recognition that this is not the same detail, take $b_{eff} = 2.5t_f$ & avoid unconservative calculation of resistance.


SAP2000
9/24/10 13:25:49
ANALYSIS WITH STABLE (FINAL) FLANGE SIZES
LS5
LS4
LS3
LS2
LS1
SAP2000 v11.0.8 - File:01f_Task1_Sect1_PrelimDim04_2RestrEdges - 3-D View - KN, mm, C Units



SAP2000
9/24/10 12:52:25
Transverse Stiffener
Plan view
N213
TS_S192
TS_S161
N181
N179
N177
TS_S160
LS5
TS_S129
N145
N143
N141
TS_S128
LS4
TS_S97
N109
N107
N105
TS_S96
LS3
TS_S65
N73
N71
N69
TS_S64
LS2
TS_S33
N37
N35
N33
TS_S32
LS1
TS_S1
N1
Z
1
F22
F12
F11
M12
M11
SAP2000 v11.0.8 - File:01f_Task1_Sect1_PrelimDim04_2RestrEdges - X-Y Plane @ Z=0 - KN, mm, C Units

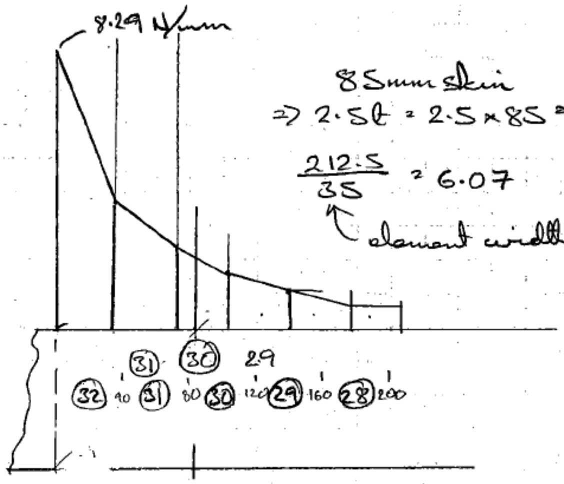


SAP2000
9/24/10 12:52:38
Longitudinal Stiffener 1
Elevation View
LS1-N42
LS1-42
LS1-40
LS1-N39
LS1-35
LS1-N34
LS1-28
LS1-N27
N35
LS1-16
LS1-N16
LS1-9
LS1-N9
LS1-4
LS1-N4
LS1-1
LS1-N1
Z
X
F11
F22
F12
M11
M22
M12
SAP2000 v11.0.8 - File:01f_Task1_Sect1_PrelimDim04_2RestrEdges - X-Z Plane @ Y=-2800 - KN, mm, C Units

420 x 25 FLANGED STIFFENER

WEB – SKIN WELDS

page 42	TS 32	8.29
		3.78
	TS 31	3.77
		2.40
	TS 30	2.39
		1.57
	TS 29	1.57
		1.04
	TS 28	1.04
		0.69
	TS 27	0.69
		0.45



85mm skin

⇒ $2.5t = 2.5 \times 85 = 212.5mm$

$\frac{212.5}{35} = 6.07$ ← element width

		F_{11}		F_{12}	
Tensile Force ts32	$\frac{8.29+3.78}{2} \times 35$ =	211 kN		$\frac{1.29+1.13}{2} \times 35$ =	42
ts31	$\frac{3.77+2.40}{2} \times 35$ =	108		$\frac{0.84+0.81}{2} \times 35$ =	29
ts30	$\frac{2.39+1.57}{2} \times 35$	69		$\frac{0.77+0.61}{2} \times 35$ =	24
29	$\frac{1.57+1.04}{2} \times 35$	46		$\frac{0.67+0.63}{2} \times 35$ =	23
28	$\frac{1.04+0.69}{2} \times 35$	30		$\frac{0.63+0.60}{2} \times 35$ =	22
27	$\frac{0.69+0.45}{2} \times 35$	20		$\frac{0.62+0.59}{2} \times 35$ =	21
		484 kN			161 kN

$\frac{484}{210} = 2.30$ kN/mm $\frac{161}{210} = 0.767$ kN/mm

On 2 No 5mm throats = 10mm

$\rightarrow \sigma_\perp = \frac{2.30 \times 10^3}{10\sqrt{2}} = 163$ MPa

$\tau_\perp = \text{——} = 163$ MPa

$\tau_\parallel = \frac{767}{10} = 77$ MPa

$\therefore \left[\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)\right]^{0.5} = \left[163^2 + 3(163^2 + 77^2)\right]^{0.5} = 352$ MPa

$\frac{f_u}{\beta_w \gamma_{M2}}$ $\frac{540}{1.0 \times 1.25} = 432 \Rightarrow \frac{D}{C} = \frac{352}{432} = 0.815 \Rightarrow$ OK for 2 throats @ 5mm

D/C reasonably < 1.0 for somewhat uncertain detail

420 x 25 FLANGED STIFFENER (cont)

VERTICAL STIFFENER – SKIN WELDS

kN/mm

Page 441 LS3_21 & 22 F11 = 2.25, F12 = 0.90 M11 = 0

Page {439 LS2_21 & 22; 442 LS4_21 & 22} F11 = 1.62, F12 = 0.64, M11 = 22.7 kN-mm/mm

Pages {438 LS1_21 & 22; 444 LS5_21 & 22} F11 = 0.855, F12 = 0.34, M11 = 32.8 kN-mm/mm

Width of stiffener = 65 mm ?

Direct Load/m on one weld = $\frac{2.25}{2} + \frac{0}{65} = 1.125$ kN/mm ; Shear = $\frac{0.90}{2} = 0.45$ kN/mm

or = $\frac{1.62}{2} + \frac{22.7}{65} = 1.159$; $\frac{0.64}{2} = 0.32$

or = $\frac{0.855}{2} + \frac{32.8}{65} < 0.932$; $\frac{0.34}{2} = 0.17$

On 1No 5mm throat

$\sigma_\perp = \frac{1.125 \times 10^3}{5\sqrt{2}} = 159$ MPa

$\tau_\perp = \text{——} = 159$ MPa

$\tau_\parallel = \frac{450}{5} = 90$ MPa

$\left[\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)\right]^{0.5}$

$= \left[159^2 + 3(159^2 + 90^2)\right]^{0.5} = 354$ MPa

$\frac{f_u}{\beta_w \gamma_{M2}} = \frac{540}{1.0 \times 1.25} = 432$ MPa

$\rightarrow \frac{D}{C} = \frac{354}{432} = 0.820 \Rightarrow$ OK

$\sigma_\perp = \frac{1.159 \times 10^3}{5\sqrt{2}} = 164$ MPa

$\tau_\perp = \text{——} = 164$ MPa

$\tau_\parallel = \frac{320}{5} = 64$ MPa

$\left[\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)\right]^{0.5}$

$= \left[164^2 + 3(164^2 + 64^2)\right]^{0.5} = 346$ MPa

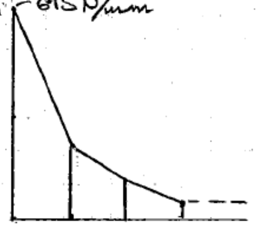
$\rightarrow \frac{D}{C} = \frac{346}{432} = 0.801 \Rightarrow$ OK

on 5mm throat welds 

390 x 20 FLANGED STIFFENER
WEB-SKIN WELDS

55 flange → average over 2.5×55 = 137.5 mm ⇒ $\frac{137.5}{35}$ = 3.93 elements

Page 445&6 → 6.15 N/mm



Direct loads F11		Shear loads F12	
$\frac{6.15+2.15}{2} \times 35$ =	145 kN	$\frac{1.00+0.86}{2} \times 35$ =	33 kN
$\frac{2.14+1.05}{2} \times 35$ =	56	$\frac{0.57+0.49}{2} \times 35$ =	19 kN
$\frac{1.05+0.50}{2} \times 35$ =	46	$\frac{0.50+0.37}{2} \times 35$ =	15 kN
0.50 × 0.93 × 35 =	16	0.40 × 0.93 × 35 =	13 kN
	263 kN		80 kN

∴ load/mm = $\frac{263}{137.5}$ = **1.913 kN/mm** $\frac{80}{137.5}$ = **0.582 kN/mm**

These values are both less than for the 420 x 25 flanged stiffener, page 473, (F11 → 2.30 & F12 → 0.767), so the 390x20 flanged stiffener is OK with the 2 throats of 5mm as page 473.

VERTICAL STIFFENER – SKIN WELDS

Stiffener min thickness = 50mm ?

Page 452 → F11 = 0.732 kN/mm, F12 = 0.298 kN/mm, M11 = 30.9 kN-mm/mm → load/mm = $\frac{0.732}{2} + \frac{30.9}{50}$ = 0.984 kN/mm; Shear = $\frac{0.298}{2}$ = 0.149 kN/mm

Page 453 → F11 = 1.353, F12 = 0.565, M11 = 21.8 → $\frac{1.353}{2} + \frac{21.8}{50}$ = 1.113; Shear = $\frac{0.565}{2}$ = 0.283

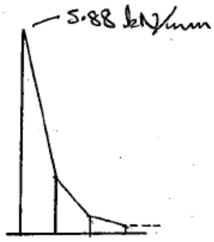
Page 455 → F11 = 1.894, F12 = 0.794, M11 = 0 → $\frac{1.894}{2}$ = 0.947; Shear = $\frac{0.794}{2}$ = 0.397

Loads/mm less than page 474 ⇒ D/C < 0.82 ⇒ OK on 5mm throat welds

<u>360 x 20 FLANGED STIFFENER</u>
<u>WEB - SKIN WELDS</u>

45 flange → average loads over 2.5×45 = 112.5mm ⇒ $\frac{112.5}{35}$ = 3.21 elements

From pages 459 and 460



	F11	F	F12	
Tensile force =	$\frac{5.88+1.59}{2}$ ×35 =	131 kN	$\frac{1.00+0.78}{2}$ ×35 =	31 kN
+	$\frac{1.58+0.65}{2}$ ×35 =	39	$\frac{0.49+0.33}{2}$ ×35 =	14
+	$\frac{0.64+0.21}{2}$ ×35 =	15	$\frac{0.39+0.25}{2}$ ×35 =	11
+	0.22 × 0.21 × 35 =	2	+ 0.31×0.21×35 =	2
		187 kN		58 kN

Average load/mm = $\frac{187}{112.5}$ = 1.66 kN/mm $\frac{58}{112.5}$ = 0.52 kN/mm

Loads/mm much less than 2.30 & 0.767 on page 473

⇒ OK on 2No welds with 5mm throats.

<u>VERTICAL STIFFENER – SKIN WELDS</u>

Stiffener min thickness = 50mm?

N/mm

Page 466 → F11 = 0.788; F12 = 0.316; M11 = 32.5 → Load/mm = $\frac{0.788}{2}+\frac{32.5}{50}$ = 1.044 Shear = $\frac{0.316}{2}$ = 0.158 N/mm

Page 467 → F11 = 1.45; F12 = 0.602; M11 = 23.1 → Load/mm = $\frac{1.45}{2}+\frac{23.1}{50}$ = 1.187 Shear = $\frac{0.602}{2}$ = 0.301 N/mm

Page 469 → F11 = 2.04; F12 = 0.847; M11 = 0 → Load/mm = $\frac{2.04}{2}$ + 0 = 1.02 Shear $\frac{0.847}{2}$ = 0.424 N/mm

Loads from pages 466 and 469 are less than from page 474 ⇒ O/C < 0.820

Loads from page 467 > page 474

⇒ O/C ≯ 0.801 × $\frac{1.187}{1.159}$ = 0.820 ⇒ OK.

Direct = 1.159, Shear = 0.32 N/mm

5.2.2 Cross Beam to Tower Leg Connection – Type 2 and 3 Diaphragms

The following calculations are presented in this section:

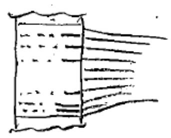
- Proportioning of the tower leg plates for the increased shear and transverse direct stresses to which they are subjected at the cross beam connections.
- Proportioning of additional transverse stiffening required on the tower leg plates to prevent buckling and to distribute the transverse direct stresses carried into the tower leg by the cross beam longitudinal stiffeners.
- Proportioning the plate diaphragm in the central tower leg cell and the plate diaphragms in the triangular tower leg cells to distribute the axial forces in the cross beams flanges to the tower leg plates.
- Proportioning of the tower top diaphragm type 3 for the additional stresses applied to the diaphragm by the main cable saddle.

INTRODUCTION

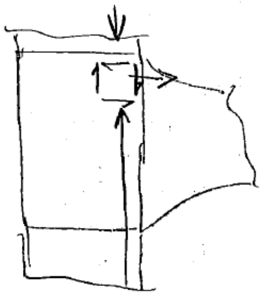
PURPOSE

To size plates and stiffeners in the walls & diaphragms, addressing the issues below:-

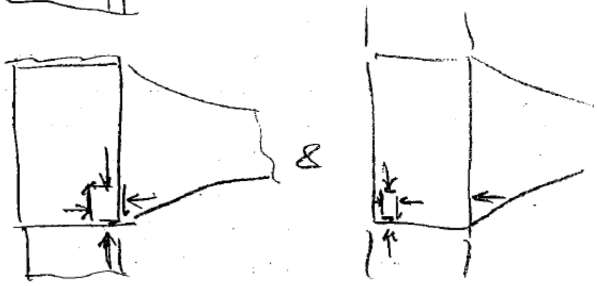
1) continuation of web stiffeners — how far?



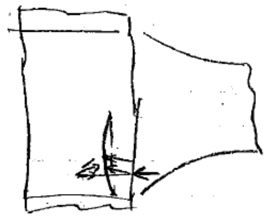
2) Von Mises $\sigma_1, -\sigma_2, +\tau$



3) Buckling from $\sigma_1, +\sigma_2, +\tau$

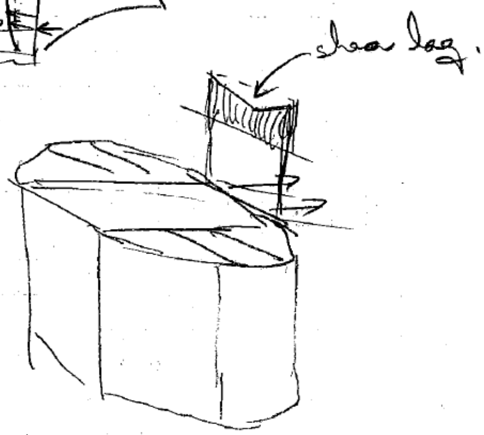


4) stability of vert stiffs from lateral compn



Need more stiffener section?

5) Shear lag in diaphragms



CONCLUSIONS → See page 601A

Page 601A

COMMENTS & CONCLUSIONS

1) Must have at least 50 pl for shear + vertical (even with almost zero horiz stress) see page 616

2) Plate H must be min 55 pl for von Mises unless stiffeners all continued

p619 → D/c for 55pl & NO stiffs = 1.09 → needs stiffs for von Mises
p620 → D/c for 55pl with stiffs = 0.97 ⇒ use 60pl for inner plate H
p621 → D/c for 60pl NO stiffs = 1.00

3) Top 5 panels need to be bounded by stiffs if 55pl on inner H – pages 617 & 619
Top 1 panel " 60pl — " — pages 617 & 621
⇒ Use 60mm pl for inner plate H to resist horizontal compression alone

4) Top 2 panels need to be bounded by stiff. for 50pl on G – page 617
Don't need horiz stiff for horiz compression if use 55pl on G – page 617
⇒ Use minimum thickness of 55mm plate for plate G

5) Need to check vertical stiffener ability to act also as transverse stiffeners to stabilise v horiz compression

6) Shear lag initial calcs ⇒ min diaphr thickness ≈ 30mm (27 is absolute min) page 621 (probably 28 is min)

7) Plate alignment issues!!!

LATEST CALCULATIONS

New calculation series started 28 Sept on page 663 using both basic calculations and COWI shell element output from IBDAS

CONCLUSIONS

1) Concludes that for 40 pl flange in cross beam, use 40 pl central diaphragm but use 60 pl for outer diaphragm & remove cut-outs from 2 vertical stiffeners on plate C closest to plate H (page 682) (page 682)
2) For plate H/G/A in height of connection, use 50 pl (pages 673–677 & 687–688) (666, 668)
3) For vertical stiffeners use 475x48 on plate G (pages 693 & 694)
& 550x55 on plate H (— " —)

DESIGN CASE LOADS

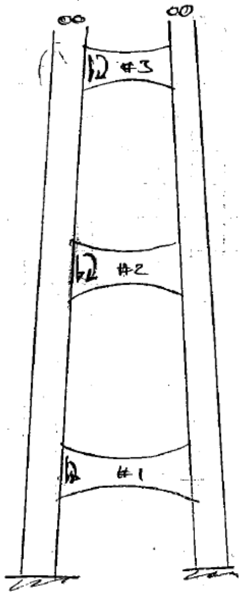
Moment & Shear
Governed by SILS and
loads @ 27 August 2010

M= 3625 MN-m
V= 145 MN

MODEL 3.2 SEP 8 / MODE 5

4186 + 1347 = 5490
165 + 54 = 219

loads @ 8 Sept 2010



Webs @ legs 40

Flange @ legs 45 + 525 x 53

4186

Web stiffs 525x53
400 x 40
300x30

The loads used in the Sept 28 calculations (not the FE output) pages 650-694 were V = 219 MN & M= 5490 MN-m from the 8 Sept 2010 figures above, because this was the latest data available when re-calculation had to recommence to meet the deadline. The loads from Model 3.3 (as in the FE output) were approx Cross-Beam #3; V= 110 MN; M= 2250 MN-m

#2 V= 140MN; M= 3550MN-m

#1 V= 105MN; M= 3100MN-m

∴ D/Cs derived from classic calculations starting with V & M are pessimistic but D/Cs or thickness derived from FE are based on Model 3.3 loads automatically.

MW3
TRANSVERSE WIND
FROM SOUTH

WIND LOADS INCREASE
BY ~15% FOR CROSS BEAMS
; 25% FOR TOWERS

1913
page 602A
2010 Oct 13

WIND LOADS DECREASED BY
~10% FOR CROSS BEAM

	Model 3.1	Model 3.2	Model 3.3
Cross Beam 1			
ULS	2663	2971	2662
SILS	3180	3559	3177
		1.12	0.89
Cross Beam 2			
ULS	3093	3539	3159
SILS	3490	4186	3712
		1.17	0.89
Cross Beam 3			
ULS	2473	2836	2521
SILS	2735	3180	2767
		1.15	0.88
Tower leg 1 Base			
ULS	2381	2988	2912
SILS	2875	3615	3505
		1.26	0.97
Tower leg 2 Base			
ULS	2799	3427	2782
SILS	3336	4113	3333
		1.23	0.81

CONNECTION TO DECK
MUST HAVE SWITCHED
FROM LEG 2 TO LEG
IN MODEL 3.3

SEPT 28 CALCULATIONS (cont)

CHECK WHICH LOADING IS DESIGN CASE

se3_6931_3 — 760 MPa at top diaph
-4 — 827 at bottom diaph } ⇒ SILS wind governs

se3_6902_3 — 166 MPa at top diaph
-4 — 185 MPa at bottom diaph

se3_6931_5 — Shear in middle ≃ 68 MPa max shear above = 200 ish
-6 — Shear in middle ≃ 300 MPa

se_6902_5 — 57 MPa in centre
-6 — 60 MPa in centre

⇒ 6931 is clearly the design case

SHEARS IN PLATES H/G/H from se3_6931_6.pdf

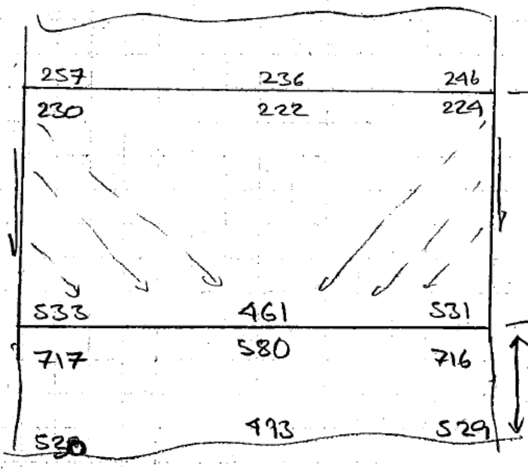
At centre of G, max τ = 310 MPa; × 30mm thickness ⇒ flow = 310×30 = 9300 N/mm

At G below upper diaphragm & above lower — " — } τ ≃ 110 MPa × 30 thick ⇒ flow = 110×30 = 3300 N/mm

At G 3.5m below upper diaphr. τ = 200 MPa × 30 → flow = 200×30 = 6000 N/mm

Along interface of outer H & skin τ = 156 MPa max × 40 thick ⇒ flow = 156×40 = 6240 N/mm max

LONGITUDINAL IN PLATE D from se4_6931_2.pdf



— Top diaphragm

— Bottom diaphragm

3m
531 × 55mm → 29200 N/mm
716 × 55mm → 39400 N/mm

SEPT 28 CALCULATIONS (cont)

LOADS

Loads used at end August (pages 611 → 649) were M = 3625, V = 145 MN

These were later superseded by next COWI analysis M = 4186 + 1347 = 5490 MN-m
Model 3.2 (Sept 8) — Mode 5 — V = 165 + 54 = 219 MN

∴ The following calculations use M = 5490 MN-m at the face of the leg + V = 219 MN

DISTRIBUTION OF FORCES ON CROSS-BEAM CROSS-SECTION

See page 664 for section used & page 665 → shear lag reduction of 0.844

Horizontal force in flange = $A_f \sigma$

σ at flange = $\frac{My}{I} = \frac{5490 \times 10^9 \times 9.917 \times 10^3}{152.7 \times 10^{12}} = 357$ MPa — see page 666

Eff Flange plate area = $0.844 \times 8000 \times 40 = 270 \times 10^3 \text{ mm}^2$

∴ Horiz Force in flange plate = σA_{eff} (reduced by shear lag) $= 357 \times 270 \times 10^3 = 96.4$ MN

σ at stiff &s = $\frac{My}{I} = \frac{5490 \times 10^9 \times 9.634 \times 10^3}{152.7 \times 10^{12}} = 346$ MPa — see page 666

6 No stiffeners area = $0.844 \times 6 \times 525 \times 53 = 141 \times 10^3 \text{ mm}^2$

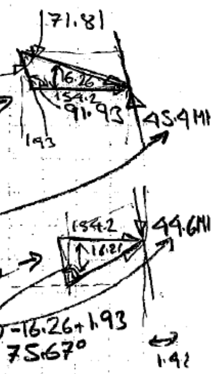
∴ Horiz Force in flange stiffeners = $\sigma A = 346 \times 167 \times 10^3 = 57.8$ MN

154.2 MN

(sketch: 14000, 50,000, 5.31)

Inclination of flanges = $\sin^{-1} \frac{14000}{50000} = 16.26°$

Inclination of tower = $\tan^{-1} \frac{\frac{1}{2}(77.662 - 52.0)}{399.0 - 18.0} = \tan^{-1} \frac{12.831}{381.0} = 1.93°$



∴ Load from bottom flange into plate D =

Sin rule → $\frac{a}{\sin A} = \frac{b}{\sin B} \Rightarrow a = b \frac{\sin A}{\sin B} =$

$= 154.2 \times \frac{\sin 16.26}{\sin 71.81} = 154.2 \times \frac{0.2800}{0.9500} = 45.4$ MN

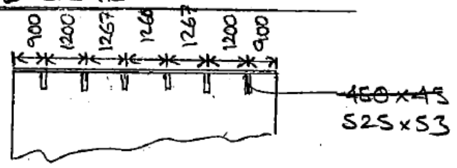
Load from top flange into plate D = $154.2 \times \frac{\sin 16.26}{\sin 75.67}$

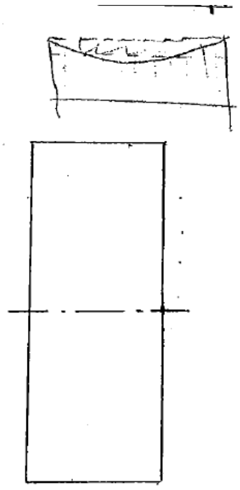
$= 154.2 \times \frac{0.2800}{0.9689} = 44.6$ MN

Additional thickness reqd = $\frac{45.4 \times 10^6}{0.844 \times 8000 \times 460} = 14.6$ mm

SEPT 28 CALCULATIONS (cont')

DIAPHRAGMS – INITIAL SHEAR LAG ESTIMATE





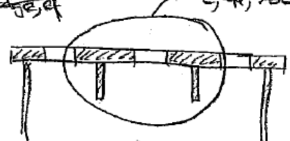
Guess flange thickness needs to be ~~30~~ 40 pL

Calculate shear lag as EN1993-1-5 #3.3

Using method of Note 3 as recommended in Note 1

$A_{eff} = A_{c,eff}\,\beta^K \geq A_{c,eff}\,\beta$ where β & K are from Table 3.1

~~$A_{c,eff}$ from #4.4 & #4.5; #4.5.1(3) → $A_{c,eff} = \rho_c A_{c,eff,loc} + \Sigma b_{edge,eff}\, t$~~ → Fig 4.4



~~Not directly relevant~~

~~(30 plate on 1267mm stiffener spacing → $\frac{b}{t} = 42$ ⇒ not fully effective)~~

$K = \alpha_0 b_0 / L_e$; $\alpha_0 = \sqrt{1 + \frac{A_{sl}}{b_0 t}} = \sqrt{1 + \frac{3 \times \cancel{450}\,525 \times \cancel{45}\,53}{4000 \times \cancel{30}\,40}} = 1.234$ ~~1.227~~

From Figure 3.1, shear force diagram is like cantilever
∴ effective length is taken from $L_e = 2L_3 = 2 \times$ cantilever length
$= 2 \times$ tower width
$= 2 \times 12m = 24m$

$\therefore K = \frac{\alpha_0 b_0}{L_e} = \frac{1.234\ (\cancel{1.227}) \times 4.0}{24.0} = 0.205$ (0.206 ≃ 0.205) ⇒ $0.02 < K < 0.70$

$\therefore \beta = \beta_2 = \frac{1}{1 + 6.0\left(K - \frac{1}{2500K}\right) + 1.6K^2} = \frac{1}{1 + 6\left(0.205 - \frac{1}{2500 \times 0.205}\right) + 1.6(0.205)^2}$
$1 + 1.218 + 0.067$

$= \frac{1}{2.286} = 0.438$

$\beta^K = (0.438)^{0.205} = 0.844$ OK = reduction of flange efficiency for shear lag.

∴ Equivalent flange area = ~~0.844 (8000×... + 6×45×450)~~ = ~~271 346~~
$0.844(8000 \times 30 + 6 \times 45 \times 450) = 305\,106\ mm^2$
Assumed in calculations: $(8000 \times 20 + 6 \times 43 \times 450) = 281\,500\ mm^2$

Note: ... ⇒ flange assumption is conservative ⇒ Need 30–27 pL mm

	E		210,000	
	nu		0	
yield stress	fy	MPa	460	
GammaM0			1.00	SILS
GammaM1			1.00	SILS
Cross-beam moment			5490	
Shear from upper leg		MN	37	page 671
height of connection		m	19.9	
Moment applied	M	MN-m	4753.7	
width of connection	la	m	12	
Shear in connection	V	N	4.0E+08	
flange thickness	tf	mm	40	
web thickness	tw	mm	40	
pl H above and below	tHout	mm	50	
pl H in connection	tH	mm	50	
pl G in connection	tG	mm	50	
face of plD to stiff CL			1175	
thickness of plD			50	
thickness of stiff			48	
CLplD to CL stiff	bH		1126	
buckling coefficient	kv		4	
sigma_cr vertically	sHv_cr		1,497	(kv*PI()^E/(12*(1-nu^2)))*(tH/bH)^2
shear lag factor	sh_lag		0.844	from page 665
gross flange breadth	bfg		8000	
max vertical stress	max_vs		150	page 670

ref page 664	space	no	b	d	ytop	yCL	Ai	Ai*yCL	yNAi	Ai*yNAi^2	Iself	bending stresses in beam MPa	deltaQi Ai*yNai	Q =sumAi*yNAi	Aveff (Itotal/2*tf)/Q/2	V/Aveff
Top flange plate		1	6752	40		20	270080	5.4E+06	9,917	2.7E+13	3.6E+07	349	2.7E+09	2.7E+09		
Top flange stiffeners		6	44.732	525		302.5	140906	4.3E+07	9,634	1.3E+13	3.2E+09	339	1.4E+09	4.0E+09	3,090,178	128
															(Itotal*no*b)/Q	
Web plate	1019	2	40	1019	0	509.5	81520	4.2E+07	9,427	7.2E+12	7.1E+09	332	7.7E+08	4.8E+09	2,595,869	153
Web stiffener	1019	2	525	53		1019	55650	5.7E+07	8,918	4.4E+12	1.3E+07	314	5.0E+08	5.3E+09		
Web plate	1022	2	40	1022	1019	1530	81760	1.3E+08	8,407	5.8E+12	7.1E+09	296	6.9E+08	6.0E+09	2,082,758	190
Web stiffener	1022	2	525	53		2041	55650	1.1E+08	7,896	3.5E+12	1.3E+07	278	4.4E+08	6.4E+09		
Web plate	1163	2	40	1163	2041	2622.5	93040	2.4E+08	7,314	5.0E+12	1.0E+10	258	6.8E+08	7.1E+09	1,754,599	226
Web stiffener	1163	2	525	53		3204	55650	1.8E+08	6,733	2.5E+12	1.3E+07	237	3.7E+08	7.5E+09		
Web plate	1349	2	40	1349	3204	3878.5	107920	4.2E+08	6,058	4.0E+12	1.6E+10	213	6.5E+08	8.1E+09	1,532,808	258
Web stiffener	1349	2	400	40		4553	32000	1.5E+08	5,384	9.3E+11	4.3E+06	190	1.7E+08	8.3E+09		
Web plate	1210	2	40	1210	4553	5158	96800	5.0E+08	4,779	2.2E+12	1.2E+10	168	4.6E+08	8.8E+09	1,421,863	279
Web stiffener	1210	2	400	40		5763	32000	1.8E+08	4,174	5.6E+11	4.3E+06	147	1.3E+08	8.9E+09		
Web plate	1250	2	40	1250	5763	6388	100000	6.4E+08	3,549	1.3E+12	1.3E+10	125	3.5E+08	9.3E+09	1,346,863	294
Web stiffener	1250	2	300	30		7013	18000	1.3E+08	2,924	1.5E+11	1.4E+06	103	5.3E+07	9.3E+09		
Web plate	1366	2	40	1366	7013	7696	109280	8.4E+08	2,241	5.5E+11	1.7E+10	79	2.4E+08	9.6E+09	1,304,938	304
Web stiffener	1366	2	300	30		8379	18000	1.5E+08	1,558	4.4E+10	1.4E+06	55	2.8E+07	9.6E+09		
Web plate	1480	2	40	1480	8379	9119	118400	1.1E+09	818	7.9E+10	2.2E+10	29	9.7E+07	9.7E+09	1,288,110	308
Web stiffener	1480	2	300	30		9859	18000	1.8E+08	78	1.1E+08	1.4E+06	3	1.4E+06	9.7E+09		
Web plate	1487	2	40	1487	9859	10602.5	118960	1.3E+09	-666	5.3E+10	2.2E+10	-23	-7.9E+07	9.6E+09	1,298,546	305
Web stiffener	1487	2	300	30		11346	18000	2.0E+08	-1,409	3.6E+10	1.4E+06	-50	-2.5E+07	8.2E+09		
Web plate	1384	2	40	1384	11346	12038	110720	1.3E+09	-2,101	4.9E+11	1.8E+10	-74	-2.3E+08	8.2E+09	1,517,327	261
Web stiffener	1384	2	300	30		12730	18000	2.3E+08	-2,793	1.4E+11	1.4E+06	-98	-5.0E+07	8.0E+09		
Web plate	1275	2	40	1275	12730	13367.5	102000	1.4E+09	-3,431	1.2E+12	1.4E+10	-121	-3.5E+08	7.9E+09	1,571,421	252
Web stiffener	1275	2	400	40		14005	32000	4.5E+08	-4,068	5.3E+11	4.3E+06	-143	-1.3E+08	7.6E+09		
Web plate	1241	2	40	1241	14005	14625.5	99280	1.5E+09	-4,689	2.2E+12	1.3E+10	-165	-4.7E+08	7.5E+09	1,672,610	237
Web stiffener	1241	2	400	40		15246	32000	4.9E+08	-5,309	9.0E+11	4.3E+06	-187	-1.7E+08	7.0E+09		
Web plate	1374	2	40	1374	15246	15933	109920	1.8E+09	-5,996	4.0E+12	1.7E+10	-211	-6.6E+08	6.8E+09	1,828,424	217
Web stiffener	1374	2	525	53		16620	55650	9.2E+08	-6,683	2.5E+12	1.3E+07	-235	-3.7E+08	6.2E+09		
Web plate	1185	2	40	1185	16620	17212.5	94800	1.6E+09	-7,276	5.0E+12	1.1E+10	-256	-6.9E+08	5.8E+09	2,154,029	184
Web stiffener	1185	2	525	53		17805	55650	9.9E+08	-7,868	3.4E+12	1.3E+07	-277	-4.4E+08	5.1E+09		
Web plate	1049	2	40	1049	17805	18329.5	83920	1.5E+09	-8,393	5.9E+12	7.7E+09	-296	-7.0E+08	4.7E+09	2,675,016	148
Web stiffener	1049	2	525	53		18854	55650	1.0E+09	-8,917	4.4E+12	1.3E+07	-314	-5.0E+08	4.0E+09		
Web plate	1033	2	40	1033	18854	19370.5	82640	1.6E+09	-9,434	7.4E+12	7.3E+09	-332	-7.8E+08	3.5E+09	3,602,784	110
	1033					19887	0	0.0E+00	-9,950	0.0E+00	0.0E+00		0.0E+00	2.7E+09		
					19887		0	0.0E+00	9,937	0.0E+00	0.0E+00		0.0E+00	2.7E+09		
Bottom flange stiffeners		6	44.732	525		19584.5	140906	2.8E+09	-9,648	1.3E+13	3.2E+09	-340	-1.4E+09	4.0E+09	3,085,892	128
Bottom flange plate		1	6752	40		19867	270080	5.4E+09	-9,930	2.7E+13	3.6E+07	-350	-2.7E+09	2.7E+09		
						Atotal	2964832		SumAy2 =	1.6E+14						
							SumAiyC	2.9E+10		SumIself =	2.2E+11					
							yNA	9,937		Itotal =	1.6E+14					

	E	
	nu	
yield stress	fy	MPa
GammaM0		
GammaM1		
Cross-beam moment		
Shear from upper leg		MN
height of connection		m
Moment applied	M	MN-m
width of connection	la	m
Shear in connection	V	N
flange thickness	tf	mm
web thickness	tw	mm
pl H above and below	tHout	mm
pl H in connection	tH	mm
pl G in connection	tG	mm
face of plD to stiff CL		
thickness of plD		
thickness of stiff		
CLplD to CL stiff	bH	
buckling coefficient	kv	
sigma_cr vertically	sHv_cr	
shear lag factor	sh_lag	
gross flange breadth	bfg	
max vertical stress	max_vs	

ref page 664	space	no	Fluxes at leg face shear sFlux	horiz hFlux	vert vFlux	Stresses in plate H =Flux/t shear tau_H	horiz sHh	vert sHv	tens+comp von Mises at plate H/D sHvM	D/C	buckling in first kh (1+(d/bH)^2)^2	sigma_cr h (kh*PI()^2*E/(12*(1-nu^2)))*(tH/d)^2 MPa	=tau+sH h+sHv	alpha_cr	alpha_ult	lambdab_pH	rho_H	rho*alpha_ult/GammaM1	D/C
Top flange plate		1	N/mm	N/mm	N/mm	flange	=Flux/tf												
Top flange stiffeners		6	5,128	13571	0	128	339	0	405	0.88			467						
						web	=Flux/tH												
Web plate	1019	2	6,104	13280	7500	122	266	150	421	0.92	3.31	1512	538	2.81	1.09	0.623	1.000	1.092	0.916
Web stiffener	1019	2																	
Web plate	1022	2	7,608	11842	7500	152	237	150	428	0.93	3.33	1511	539	2.80	1.07	0.619	1.000	1.074	0.931
Web stiffener	1022	2																	
Web plate	1163	2	9,031	10303	7500	181	206	150	440	0.96	4.27	1499	537	2.79	1.05	0.612	1.000	1.045	0.957
Web stiffener	1163	2																	
Web plate	1349	2	10,338	8534	7500	207	171	150	453	0.99	5.93	1546	527	2.93	1.01	0.588	1.000	1.015	0.985
Web stiffener	1349	2																	
Web plate	1210	2	11,144	6732	7500	223	135	150	458	1.00	4.64	1505	508	2.96	1.00	0.582	1.000	1.004	0.996
Web stiffener	1210	2																	
Web plate	1250	2	11,765	4999	7500	235	100	150	462	1.00	4.98	1513	485	3.12	1.00	0.565	1.000	0.995	1.005
Web stiffener	1250	2																	
Web plate	1366	2	12,143	3156	7500	243	63	150	461	1.00	6.11	1554	456	3.41	1.00	0.541	1.000	0.997	1.003
Web stiffener	1366	2																	
Web plate	1480	2	12,301	1152	7500	246	23	150	456	0.99	7.44	1612	419	3.85	1.01	0.512	1.000	1.008	0.992
Web stiffener	1480	2																	
Web plate	1487	2	12,203	938	7500	244	19	150	452	0.98	7.53	1616	413	3.91	1.02	0.510	1.000	1.018	0.983
Web stiffener	1487	2																	
Web plate	1384	2	10,443	2960	7500	209	59	150	407	0.89	6.30	1562	418	3.74	1.13	0.550	1.000	1.130	0.885
Web stiffener	1384	2																	
Web plate	1275	2	10,084	4833	7500	202	97	150	410	0.89	5.21	1520	448	3.39	1.12	0.575	1.000	1.121	0.892
Web stiffener	1275	2																	
Web plate	1241	2	9,474	6605	7500	189	132	150	409	0.89	4.90	1511	472	3.20	1.12	0.592	1.000	1.124	0.890
Web stiffener	1241	2																	
Web plate	1374	2	8,666	8447	7500	173	169	150	408	0.89	6.20	1557	492	3.16	1.13	0.597	1.000	1.127	0.887
Web stiffener	1374	2																	
Web plate	1185	2	7,356	10249	7500	147	205	150	400	0.87	4.44	1501	502	2.99	1.15	0.620	1.000	1.149	0.870
Web stiffener	1185	2																	
Web plate	1049	2	5,924	11823	7500	118	236	150	395	0.86	3.49	1505	505	2.98	1.16	0.625	1.000	1.165	0.859
Web stiffener	1049	2																	
Web plate	1033	2	4,398	13289	7500	88	266	150	395	0.86	3.39	1508	504	2.99	1.16	0.623	1.000	1.164	0.859
	1033																		
Bottom flange stiffeners		6	5,135	13591	0	128	340	0	406	0.88			468						
Bottom flange plate		1																	

SEPT 28 CALCULATIONS (cont)

DISTRIBUTION OF FORCES IN PLATES H/G/H

1) Find shear assuming average shear flux for simplicity

Average shear flux = $\bar{q}$

Assume all moment from flanges (simplification)

Assume moment applied equally to each plane of H/G/H

$\therefore \bar{q} \times 12 \times 19.9 = 5490/2$ MN-m

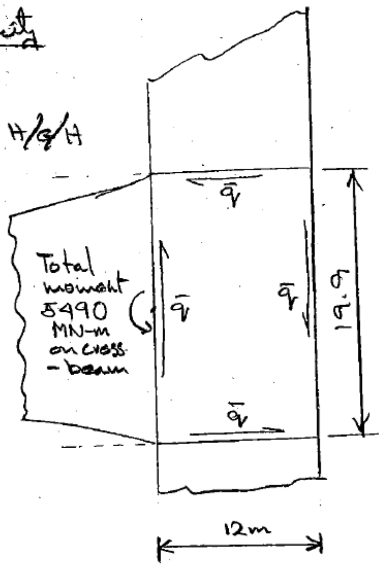
$\therefore \bar{q} = \frac{5490/2}{12 \times 19.9} = 11.49$ MN/m in each plane

$= 11.49 \times \frac{10^6 N}{10^3 mm} = 11490 N/mm$

Max shear stress $\frac{f_y}{\sqrt{3}}/\gamma_{M0} = \frac{460/\sqrt{3}}{1.05} = 253$ MPa

$\therefore$ Minimum plate thickness $= \frac{11490}{253} = 45.4$ mm

Will need to be much thicker due to coincident vertical and horizontal normal stresses.



2) Find von Mises stresses in inner pl H — web stiffeners continue here

See page 666 — for 50 mm pl H above & below → t → 80 mm

Better increase vertical stiffeners

3) Find increase in plate thickness required for plates G&H from COWI FE

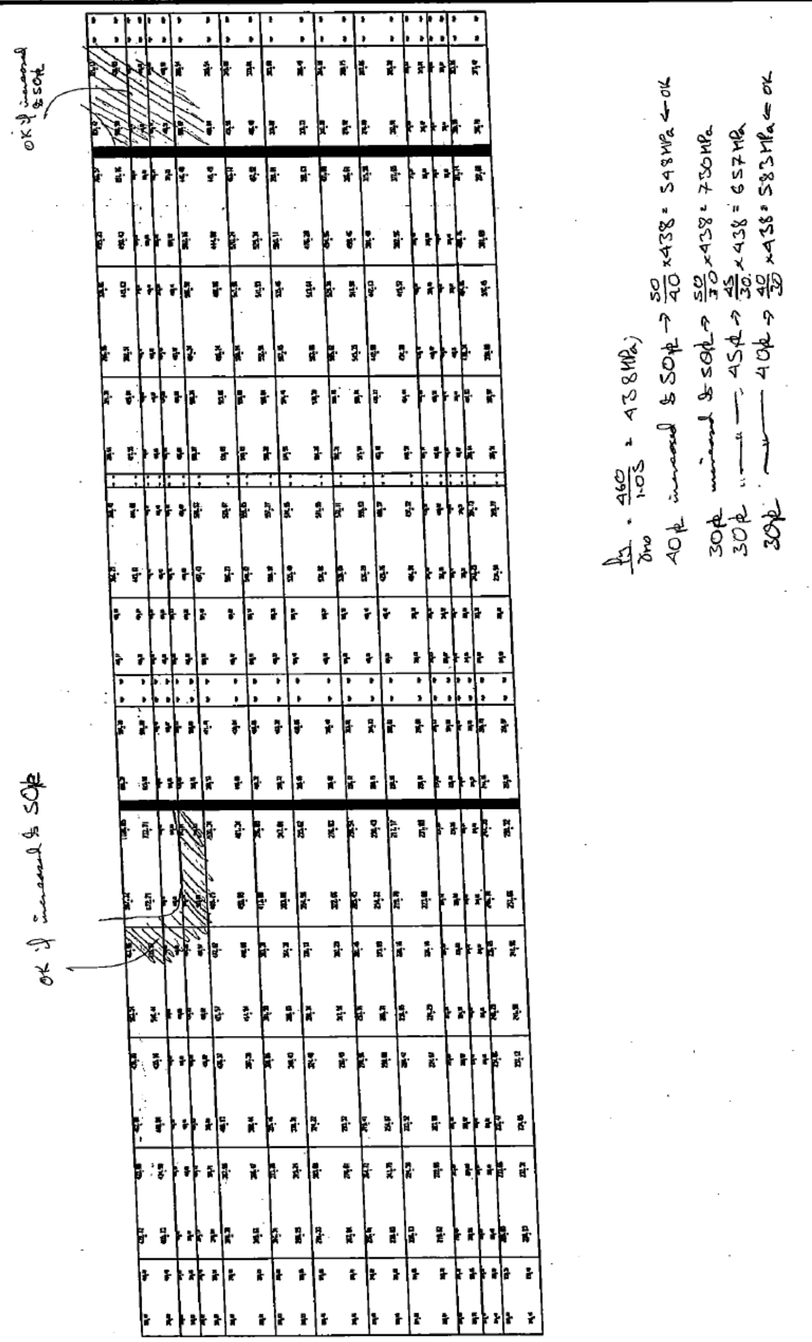
See page 668 → 50 mm OK for pl H
& 40 mm OK for pl G } but ignores increased force attracted by increased thickness

4) Find increase in plate thickness for plate D from COWI FE

See page 669 → 67 mm required below cross beam

Better increase vertical stiffeners.

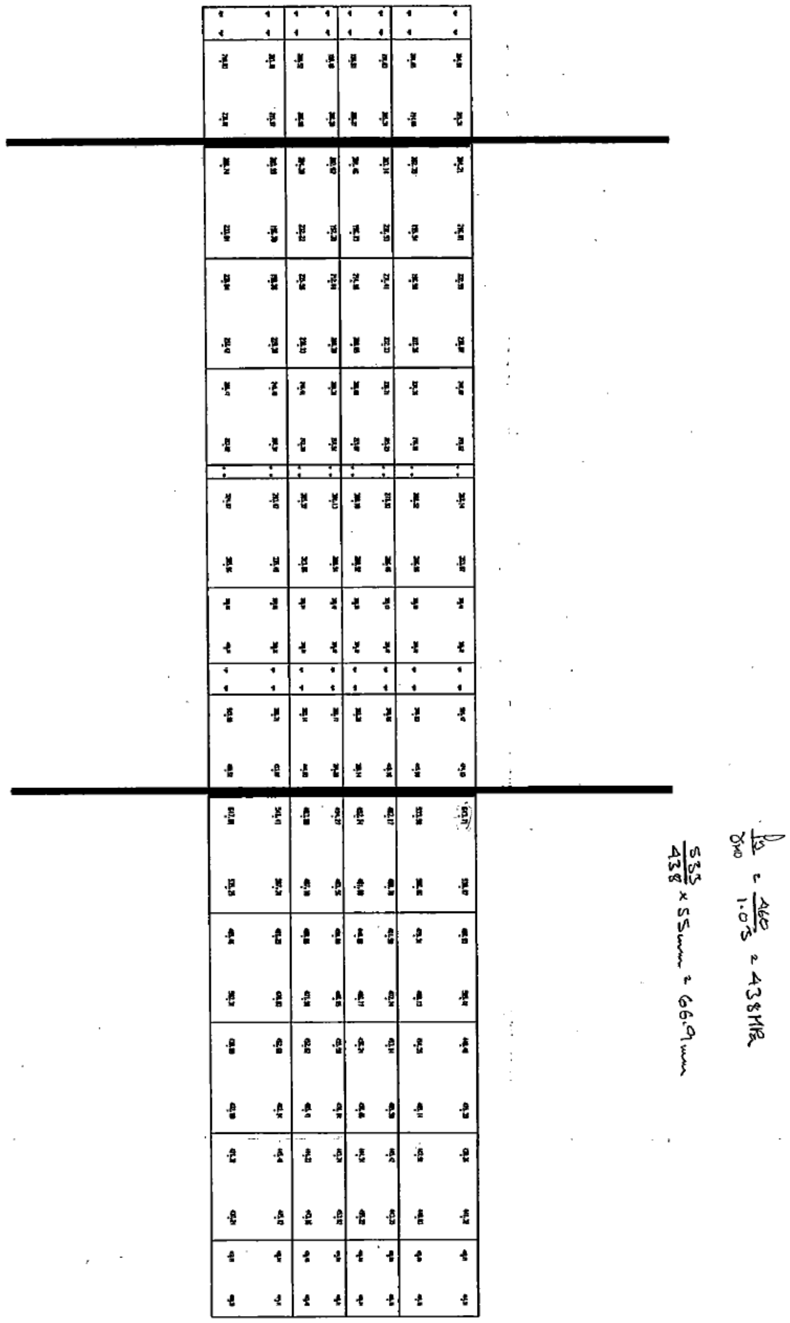
COWI Author: scc Date: 27-sep-10 12:27	Job no: 72889-a	Bridge across Strait of Messina Progetto Definitivo. Model ver. 3.3	Page:



seranl15<'c:\ibdasjobs\72889\messina-v3_shell\g6_s'320<'g6c1'4<global (phase1100 at time=100.0days) case6931 (ULS; SILS load
Wall H/G/H (sidespan side) (left view) Von-Mises_bot [MPa] (max) (elastic)

IBDAS V1.10-2 pc23562 c:\ibdasjobs\72889\messina-v3_shell\shelloutput_tower

COWI Author: scc Date: 27-sep-10 12:27	Job no: 72889-a	Bridge across Strait of Messina Progetto Definitivo. Model ver. 3.3	Page:



seranl15<'c:\ibdasjobs\72889\messina-v3_shell\g6_s'320<'g6c1'4<global (phase1100 at time=100.0days) case6931 (ULS, SLS load
Wall D (cross beam side) (front view) Von-Mises_bot [MPa] (max) (elastic)

IBDAS V1.10-2 pc23562 c:\ibdasjobs\72889\messina-v3_shell\shelloutput_tower

Sept 28 Calculations (cont)

SEPT 28 CALCULATIONS (cont)

LEG STRESSES FROM TRANSVERSE BENDING

I_{minor} #14 Sicilia

Approximate section properties

A) $2 \times 95 \times (4000)^3/12$ = 1.0×10^{12} mm^4

B) $4 \times 60 \times 3500 \times 3000^2$ = 7.6×10^{12}

$4 \times 3 \times 700 \times 70 \times 3000^2$ = 5.3×10^{12}

C) $4 \times 45 \times 3500 \times 5000^2$ = 15.8×10^{12}

$4 \times 2 \times 675 \times 68 \times 5000^2$ = 9.2×10^{12}

D) $2 \times 55 \times 6000 \times 6000^2$ = 31.7×10^{12}

$2 \times 5 \times 650 \times 65 \times 5675^2$ = 13.6×10^{12}

E) $4 \times 65 \times 3000 \times 2000^2$ = 3.1×10^{12}

$4 \times 3 \times 600 \times 60 \times 1700^2$ = 1.2×10^{12}

F) $4 \times 30 \times 3000 \times 2000^2$ = 1.4×10^{12}

$4 \times 2 \times 500 \times 50 \times 1700^2$ = 0.6×10^{12}

G) $2 \times 30 \times 4000^3/12$ = 0.3×10^{12}

H) $4 \times 45 \times 4000 \times 4000^2$ = 11.5×10^{12}

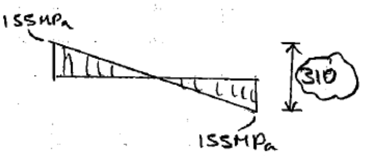
$4 \times 2 \times 550 \times 55 \times 4000^2$ = 3.9×10^{12}

106.2×10^{12} mm^4

Assume cross beam moment shared 50% above & 50% below

$\therefore M = \frac{5490}{2}$ ← Page 663 $= 2745$ MN-m

$\therefore \hat{\sigma}_b = \frac{My}{I} = \frac{2745 \times 10^9 \times 6000}{106.2 \times 10^{12}} = 155$ MPa



$\therefore$ Max compressive stress on inside tensile corner

$= 460 - 155 - 155 = 150$ MPa

SILS → $\gamma_M = 1.0$

This stress is much lower than on pages 612 → 621, so the von Mises stresses are lower.

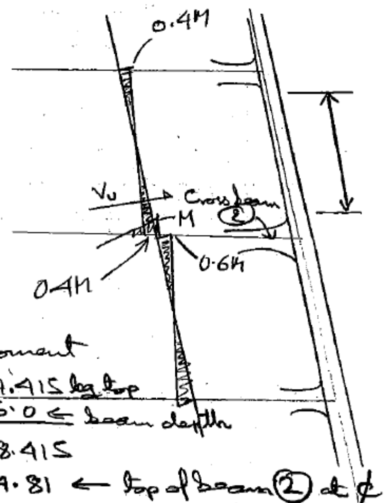
SEPT 28 CALCULATIONS (cont)

DISTRIBUTION OF SHEAR

EFFECT OF SHEAR IN UPPER SECTION OF LEG

Assume cross-beam end moment is divided 40% : 60% into upper and lower leg sections

Assume moment at underside of top cross-beam is same as above cross beam (2)



∴ Shear in upper section of leg from cross-beam moment

$V_U = \frac{M_{top} + M_{bottom}}{distance}$ where distance = 384.415 leg top
− 16.0 ← beam depth
368.415
− 254.81 ← top of beam (2) at ℄
+ 5.0 m ← height to top of leg
118.605 m

$= \frac{0.4M + 0.4M}{118.6} = \frac{0.8M}{118.6}$

$= \frac{0.8 \times 5490}{118.6} =$ 37.0 MN

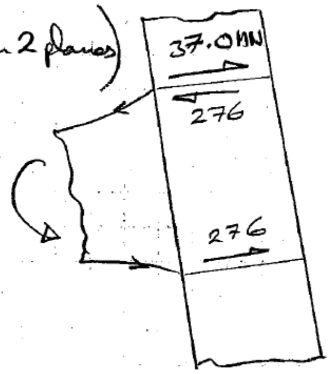
∴ reduction of shear flux $= \frac{37.0 \times 10^6}{12.0 \times 10^3} = 3.08$ kN/mm (on 2 planes)

If all moment carried in cross beam flanges

average shear $= \frac{5490}{19.9} = 276$ MN

less shear in upper leg = − 37 MN

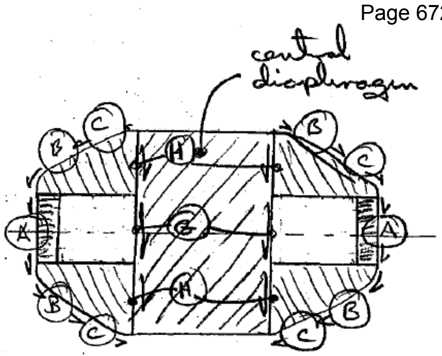
239 MN



SEPT 28 CALCULATIONS (cont)

DISTRIBUTION OF SHEAR (cont)

DISTRIBUTION TO PLATES A,B,C



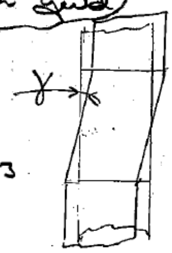
Shears in ABC will be determined
(1) by the stiffness and strength of the diaphragms connecting A,B,C to the central diaphragm

(2) by the stiffness in shear of plates ABC compared with H,G,H

Design for plates H,G,H to resist shear stress = 0.6 shear yield

$= 0.6 \times \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = 0.6 \times \frac{460}{\sqrt{3}} = 159\,MPa$

1.0 for SILS weld



∴ Shear strain $\gamma = \frac{\tau}{G} = \frac{159}{80.8 \times 10^3} = 1.968 \times 10^{-3}$

$G = \frac{E}{2(1+\nu)} = \frac{210000}{2.6}$

∴ Shear deformation over height of connection $= 19.9 \times 10^3 \times 1.968 \times 10^{-3} = 39.2\,mm$

∴ Deformation at top and bottom $= \frac{39.2}{2} = 19.6\,mm$

Assume plates A, B, C resist shear stress at 0.2 shear yield

∴ Deformation at top and bottom $= \frac{0.2}{0.6} \times 19.6 = 6.5\,mm$

∴ Displacement of plate A relative to plates H, G, H = 19.6 mm − 6.5 mm = 13.1 mm

Need stiffness of outer diaphragms to be sufficient to develop resistance to shear from (i) connection (ii) upper leg in deflection of 13.1 mm.

SEPT 28 CALCULATIONS (cont)

DISTRIBUTION OF SHEAR (cont)

DISTRIBUTION TO PLATES A,B,C (cont)

Shear flow in plate A = 0.2 × shear yeild × thickness

$= 0.2 \times \frac{f_y}{\sqrt{3}} \times 95 = 0.2 \times 266 \times 95 = 5046$ N/mm $= 5.05$ kN/mm

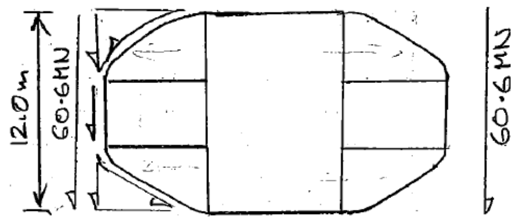
By "complementary shears", to maintain equilibrium, the same shear flow will be resisted by plates B & C

The vector sum of these forces

$= 12.0 \times 10^3 \times 5.05$ kN

$= 12.0 \times 5.05$ MN

$= 60.6$ MN from one side



∴ Total shear in all plates A, B, C = 2×60.6 = 121 MN

Net shear across connection from page 671 = 239 MN

Less Shear in plates A, B, C = −121 MN

∴ Shear in plates H/G/H = 118 MN

∴ Plate thickness to carry 118 MN at 0.6 shear yeild (as page 672)

0.6 shear yeild $= 0.6 \times \frac{f_y}{\sqrt{3}} = 159$ MPa

∴ Plate thickness required $= \frac{118 \times 10^6}{2 \times 159 \times 12000} = 31$ mm

2 planes of plate H, G, H

But need thicker to resist in-plane stresses from cross beam web

(i) for von Mises

(ii) for transverse buckling

⇒ probably need 50 ₱ minimum
agrees with thickness deduced from COWI shell elements on page 668

SEPT 28 CALCULATIONS (cont)

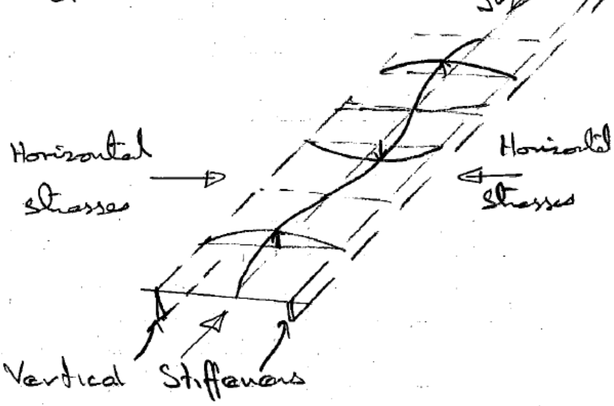
BUCKLING RESISTANCE WITHOUT HORIZ STIFFENERS

MODES OF BUCKLING

(1) BUCKLING BY "DISHING"

Buckling of plate between vertical stiffeners has been checked in the standard tower leg calculations

The buckling mode is "dishing" on alternate side

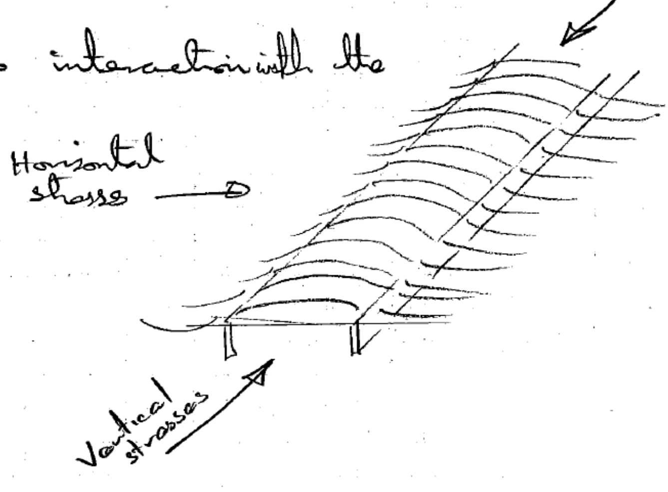


The coexistence of horizontal stresses increases the instability of this mode of buckling only on the basis of square "dishing".

(2) BUCKLING BY HORIZONTAL STRUT BUCKLING

The other possible form of buckling is like strut buckling

In this case, there is no interaction with the vertical stresses



SEPT 28. CALCULATIONS (cont)

BUCKLING RESISTANCE WITHOUT HORIZONTAL STIFFENERS (cont)

EFFECT OF SHEAR STRESSES

The plates are so thick compared with the stiffener $\frac{1200}{50} = 24$ spacing that there is no buckling effect from shear buckling but the shear reduces the effective yield stress (by von Mises)

SEPT 28 CALCULATIONS (cont)

BUCKLING RESISTANCE WITHOUT HORIZ STIFFENERS (cont)

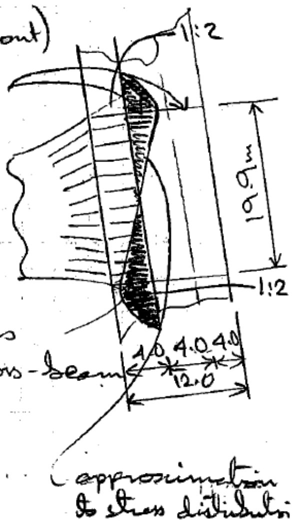
HORIZONTAL STRESSES

Plate G has no horiz stiffeners
Stresses disperse to approx zero at 45° from diaphragm
Find stress at plate G
Assume moment from cross-beam web reduced to $\frac{2}{3}$
Assume stress disperses on section at 1:2

$\therefore$ Depth of effective section

$= \frac{4}{2} + 19.9 + \frac{4}{2} = 23.9\text{m}$

Assume 50 mm plate



$\therefore$ Inertia of section (reduced for shear lag)

$=$ Flanges: $\rightarrow 2 \times \frac{2}{3} \times 6752 \times 40 \times 9.95^2 \times 10^6 = 35.6 \times 10^{12}\ \text{mm}^4$
Stiffeners: $\rightarrow 2 \times \frac{2}{3} \times 44.73 \times 525 \times 6 \times 9.65^2 \times 10^6 = 17.5 \times 10^{12}$
Webs: $\rightarrow 2 \times 50 \times (23.9)^3 \times 10^9 / 12 = 113.8 \times 10^{12}$
$166.9 \times 10^{12}\ \text{mm}^4$

Assume moment decreases linearly between faces of leg
Moment resisted $= \frac{2}{3} \times 5490 = 3660 \times 10^9$

$\therefore$ max stress $\frac{My}{I} = \frac{3660 \times 10^9 \times 9.95 \times 10^3}{166.9 \times 10^{12}} =$ (218 MPa)

HORIZONTAL STRUT BUCKLING

Check if 50 mm thickness is sufficient for 218 MPa applied

uniform compression: $\sigma_{cr} = \frac{1.0\,\pi^2 E}{12(1-\nu^2)}\left(\frac{50}{1200}\right)^2 = 330\ \text{MPa} \rightarrow \bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{460}{330}} = 1.394$

$\therefore \rho = \frac{\lambda_p - 0.22}{\lambda_p^2} = \frac{1.394 - 0.22}{(1.394)^2} = 0.604$

$\therefore \rho f_y / \gamma_m = \frac{0.604 \times 460}{1.0} = 278\ \text{MPa} \rightarrow \frac{D}{C} = \frac{218}{278} = 0.784$

(S 1.5; t needed; t provided; page 673)

From page 673, coincident shear $\left(\frac{31}{50} \times 159\right)$ (99) reduces effective yield stress by von Mises

$\sigma_1 = \sqrt{f_y^2 - 3\tau^2} = \sqrt{460^2 - 3 \times 99^2} = 427\ \text{MPa}$

$\therefore \rho f_{y\,reduced} / \gamma_m = \frac{0.604 \times 427}{1.0} = 258\ \text{MPa} \rightarrow \frac{D}{C} = \frac{218}{258} = 0.844 \Rightarrow \text{OK}$

SEPT 28 CALCULATIONS (cont)

BUCKLING RESISTANCE WITHOUT HORIZONTAL STIFFENERS (cont)

BUCKLING BY "DISHING"

On a square pattern of buckling
For uniaxial compression, $k = 4$

For equal compression in 2 directions $k = 2$

Simplest solution for different stresses is to sum the stresses from the two directions and proportion k from the sum

Vertical stress at edge of G (at centre of 1200mm vert stiff spacing)

$$= 460 - \frac{\left(8.0 - \frac{1.2}{2}\right)}{12.0} 310 = 460 - 191 = 269 \text{ MPa}$$

f_y ↗   page 670 ↗

Horizontal stress at edge of G = 218 MPa ← page 676

487 MPa

$$\therefore k_{vert} = \frac{269}{487} \times 4 = 2.21 \rightarrow \sigma_{cr.Vert} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 = \frac{2.21\,\pi^2\, 210000}{12 \times 0.91} \times \left(\frac{50}{1200}\right)^2 = 728 \text{ MPa}$$

$$k_{horiz} = \frac{218}{487} \times 4 = 1.79$$

Check according to EN1993-1-5 #10(2)

α_{ult} from von Mises stress:-

$$\sigma_0^2 = \sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2 + 3\tau^2 = 269^2 + 218^2 - 269 \times 218 + 3(99)^2 \quad \text{(page 676)}$$

$$\sigma_0 = \sqrt{72361 + 47524 - 58642 + 29403} = \sqrt{90646} = 301 \text{ MPa}$$

$$\therefore \alpha_{ult,k} = \frac{460}{301} = 1.528$$

$$\alpha_{cr} = \frac{728}{269} = 2.71$$

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = \sqrt{\frac{1.528}{2.71}} = 0.751$$

$$\therefore \rho = \frac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2} \leftarrow \text{uniform compression} = \frac{0.751 - 0.22}{(0.751)^2} = 0.941$$

$\therefore$ Max resistance $= \rho f_y = 0.941 \times 460 = 433$ MPa

$$\therefore \frac{D}{C} = \frac{269}{433} = 0.621$$

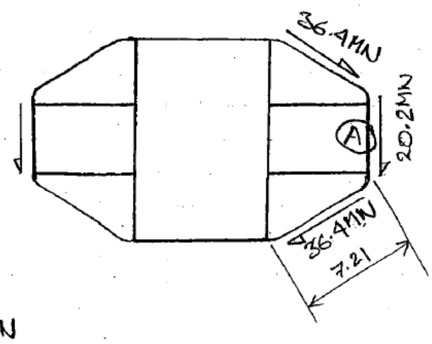
OUTER DIAPHRAGMS

SHEARS FROM CONNECTION

Shear along diagonal sides
= Shear flow in plates A = 5.05kN/mm (page 673)

∴ Total = 5.05 × 7210 = 36.4 MN

Shear along plate A = 5.05 × 4000 = 20.2 MN



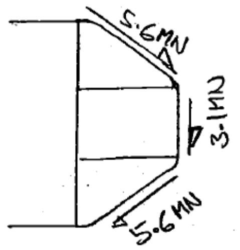
SHEARS FROM UPPER SECTION OF LEG

Page 671 → shear in upper section of leg ≏ 37.0 MN
Page 673 → shear in plates A, B, C (both sides) = 121 MN compared with 118 MN in G/H

One side Shear in plate A, B, C from upper section ≏ $\frac{1}{2} \times \frac{121}{239} \times 37.0 = 9.4$ MN

∴ Additional shear in plate A = $\frac{4}{12} \times 9.4 = 3.1$ MN

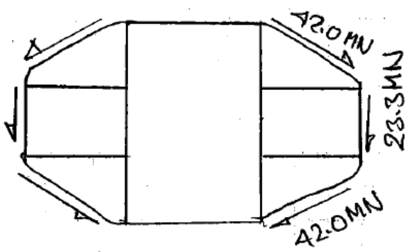
Additional shear in diagonal = $\frac{7.21}{12} \times 9.4 = 5.6$ MN



TOTAL FORCES ON DIAPHRAGMS

20.2 + 3.1 = 23.3 MN

36.4 + 5.6 = 42.0 MN



SEPT 28 CALCULATIONS (cont)

OUTER DIAPHRAGMS

SHEAR RESISTANCE

From | & sections dimensions | on page 680

Note: The diaphragms above and below the flanges of the cross beams share the load. This can be seen from the output from the COWI shell elements on page
Reviewing the sharing for the thicker plate at the flange level, the sharing will be taken as 10%, 80%, 10%

Section ①①

Resisting 80% total
Applied shear $= 0.8 \times \frac{7234}{4000} \times 23.3 = 33.7$ MN

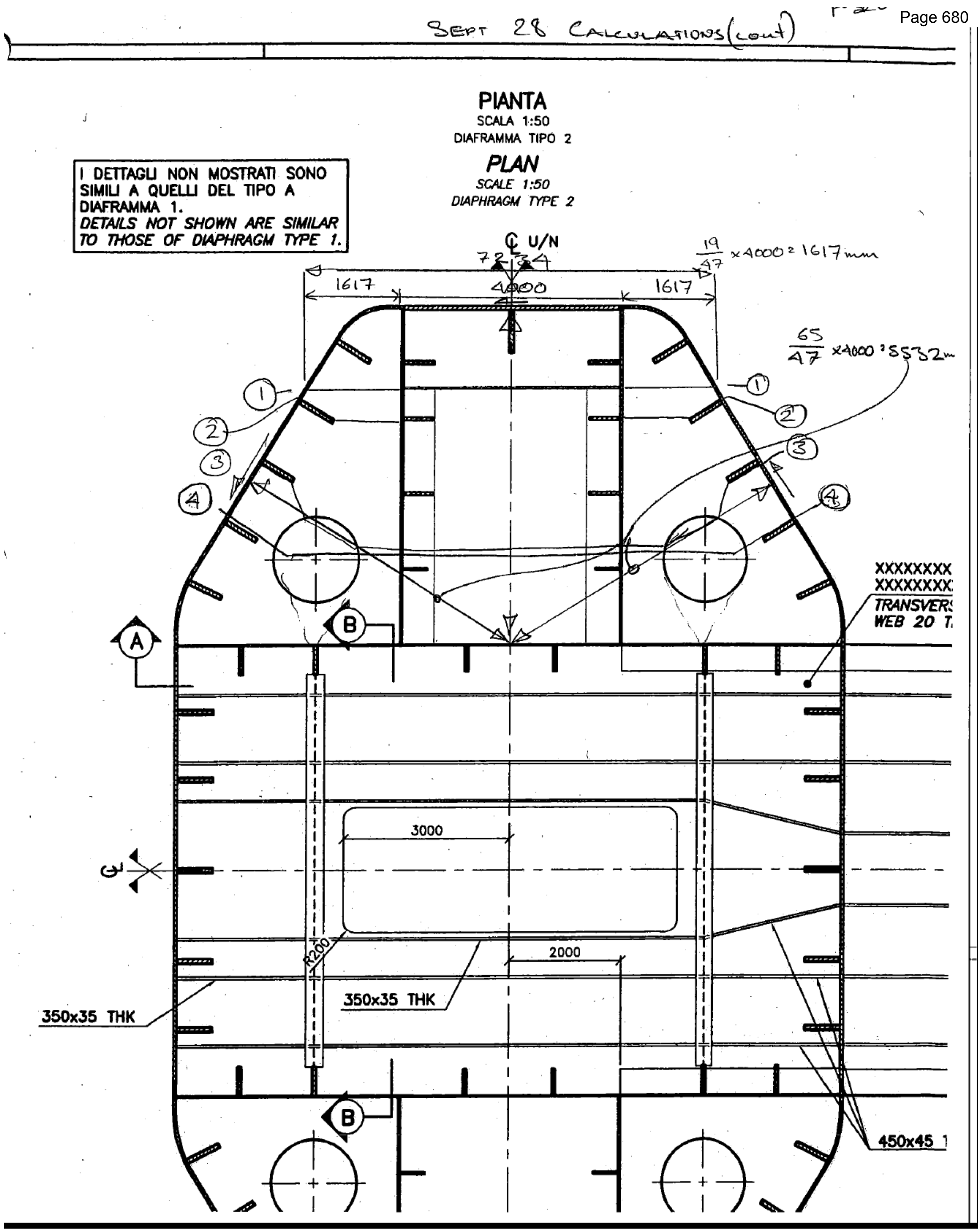
$\frac{f_y}{\sqrt{3}} = \frac{460}{\sqrt{3}}$

∴ Average shear stress $= \frac{33.7 \times 10^6}{2 \times 40 \times 1617} = 261$ MPa $\Rightarrow \frac{D}{C} = \frac{261}{266} = 0.981$

Using $\eta = 1.2$ as EN 1993-1-5 #5.1 & #5.2 $\frac{D}{C} = \frac{261}{1.2 \times 266} = \frac{261}{319} = 0.818$

Sections ②② & ③③ & ④④

Shear resistance is reduced by cut-outs at stiffeners and by access holes through plate (reduced to 1.0m dia from 1.5m dia shown on page 680), but this loss of resistance is compensated for by the slope of plates B & C and the consequent "shear" component of the direct stresses in those plates



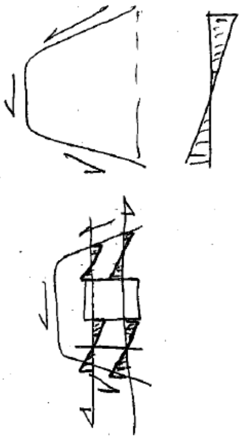
Sept 28 Calculations (cont)
PIANTA
SCALA 1:50
DIAFRAMMA TIPO 2
PLAN
SCALE 1:50
DIAPHRAGM TYPE 2
I DETTAGLI NON MOSTRATI SONO SIMILI A QUELLI DEL TIPO A DIAFRAMMA 1.
DETAILS NOT SHOWN ARE SIMILAR TO THOSE OF DIAPHRAGM TYPE 1.
℄ U/N
1617
4000
1617
19/47 x4000 = 1617mm
65/47 x4000 = 5532mm
3000
2000
350x35 THK
350x35 THK
R200
450x45
TRANSVERS
WEB 20

SEAT 28 CALCULATIONS (cont)

OUTER DIAPHRAGMS (cont)

BENDING RESISTANCE

Lever arm to diagonals from page 680 = 5532 mm



$M = 2 \times 42.0 \times 5.53 = 465$ MN-m
$+ \; 23.3 \times 6.0 = 140$ MN-m
605 MN-m

1) Assume only 10t of skin effective
$= 10 \times 45 = 450$ mm each side

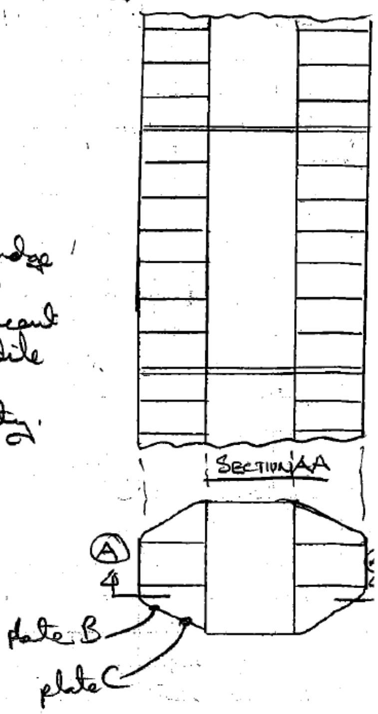
(central diaphragm)

$I = 2 \times 900 \times 45 \times 6000^2 = 2.92 \times 10^{12}$ mm^4
$+ \; 2 \times 4000 \times 40 \times 4000^2 = 5.12 \times 10^{12}$
$+ \; 2 \times 40 \times (4000)^3/12 = 0.43 \times 10^{12}$
8.47×10^{12} mm^4

$\therefore \hat{\sigma} = \frac{My}{I} = \frac{605 \times 10^9 \times 6 \times 10^3}{8.47 \times 10^{12}} = 429$ MPa — possible

The actual behaviour will include a considerable height of the skin because the diaphragms above and below participate and the plane of plate B & C are about 7.0 m deep, so there can be no appreciable movement on one diaphragm level without mobilising 2 or 3 diaphragms above and below ⇒ in-plane stresses are low.

This agrees with experience on Humber Bridge deck fastening to adjust the alignment of certain diaphragms relative to the adjacent diaphragms. Despite the rounded corner profile in very thin plate, the stiffness of the structure was enormous, ≫ jack capacity.



Page 682

SEPT 28 CALCULATIONS (cont)

OUTER DIAPHRAGMS (cont)

SUMMARY

1) COWI shell element output, pages 683 (thickness) & 684 (stresses), confirm that plates A (and also B & C) share the shear in the cross-beam to leg connections. This is shown by the rise of shear stress in the zone between the top & bottom diaphragms.

2) COWI shell element output page 684 also confirms that the diaphragms above and below the cross-beam flanges contribute to carrying these shears to plate A. This is shown by the change in shears in the panels above and below the flanges.

3) The von Mises stresses in the COWI shell elements of the diaphragms are shown on pages 685 & 686 (max stress = 1001 MPa on page 686) which are for 20 mm plate diaphragm.

The maximum stress appears at the rounded corner of plate C – plate D and is about 980 MPa

This diaphragm will be increased in thickness, so the sharing with the diaphragms above and below will be less than from the COWI model

From page 684, sharing was approx ⇒ 20%; 60%; 20% (step in τ ≈ 10 30 10)

von Mises is influenced by shear and direct stresses
Direct transverse stresses are a function of skin and diaphragm

∴ Assume sharing changes to 10% 80% 10%

$$\Rightarrow \text{Increase in loads} = \frac{80\%}{60\%} = 1.33$$

$$\therefore \text{Plate thickness required} = \text{COWI } t \times \frac{\text{COWI von Mises}}{f_y/\gamma_{M}} \times 1.33$$

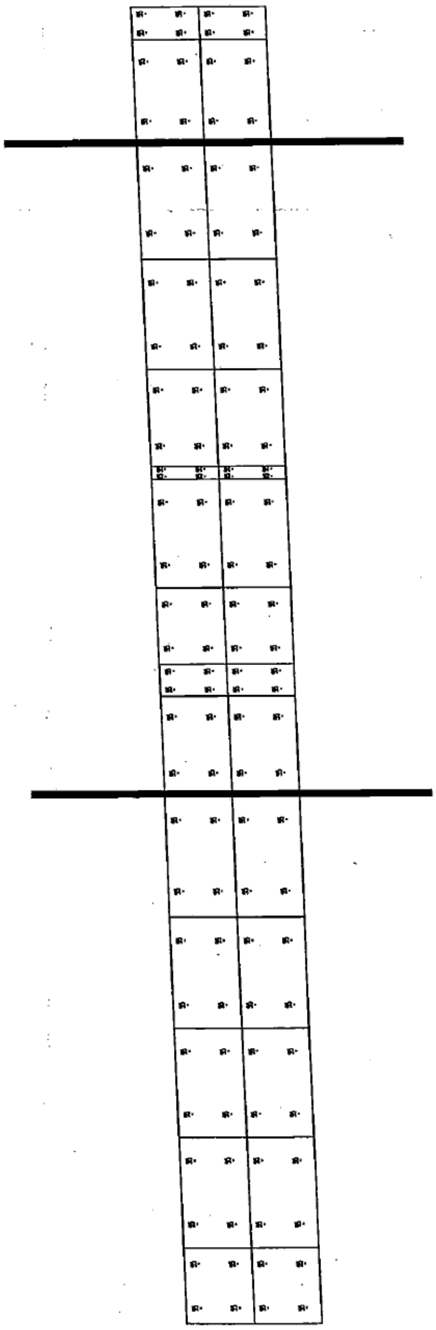
1.0 for SILS

$$= 20 \times \frac{1001}{460} \times 1.33 = 57.9 \text{ mm plate.}$$

⇒ Need 60 mm plate for outer diaphragms (without cut-outs to interrupt stress paths)

Central diaphragm is sized by flange stress → Thickness $= \frac{881}{460} \times 20 = 38.3$ mm ⇒ 40 plate.

COWI Author: scc Date: 30-sep-10 10:55	Job no: 72889-a	Bridge across Strait of Messina Progetto Definitivo. Model ver. 3.3	Page:



seranl19<'c:\ibdasjobs\72889\messina-v3_shell\g6_s'320<'g6c1'4<global (phase1100 at time=100.0days)
Wall A (Sidespan side) (left view) T [mm]

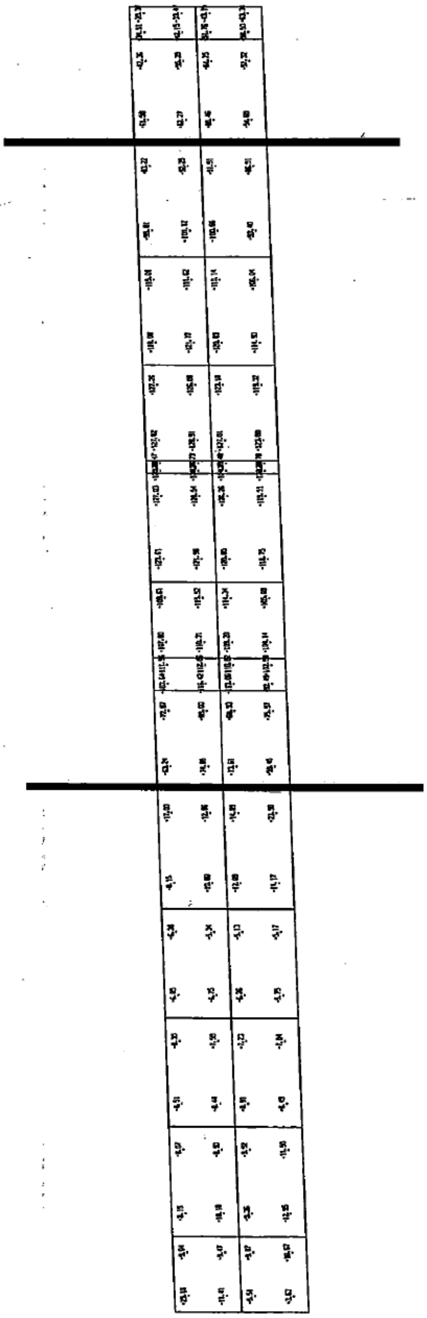
(T)
+9.50E+01
+9.50E+01

IBDAS V1.10-2 pc23562 c:\ibdasjobs\72889\messina-v3_shell\shelloutput_tower

Page 684

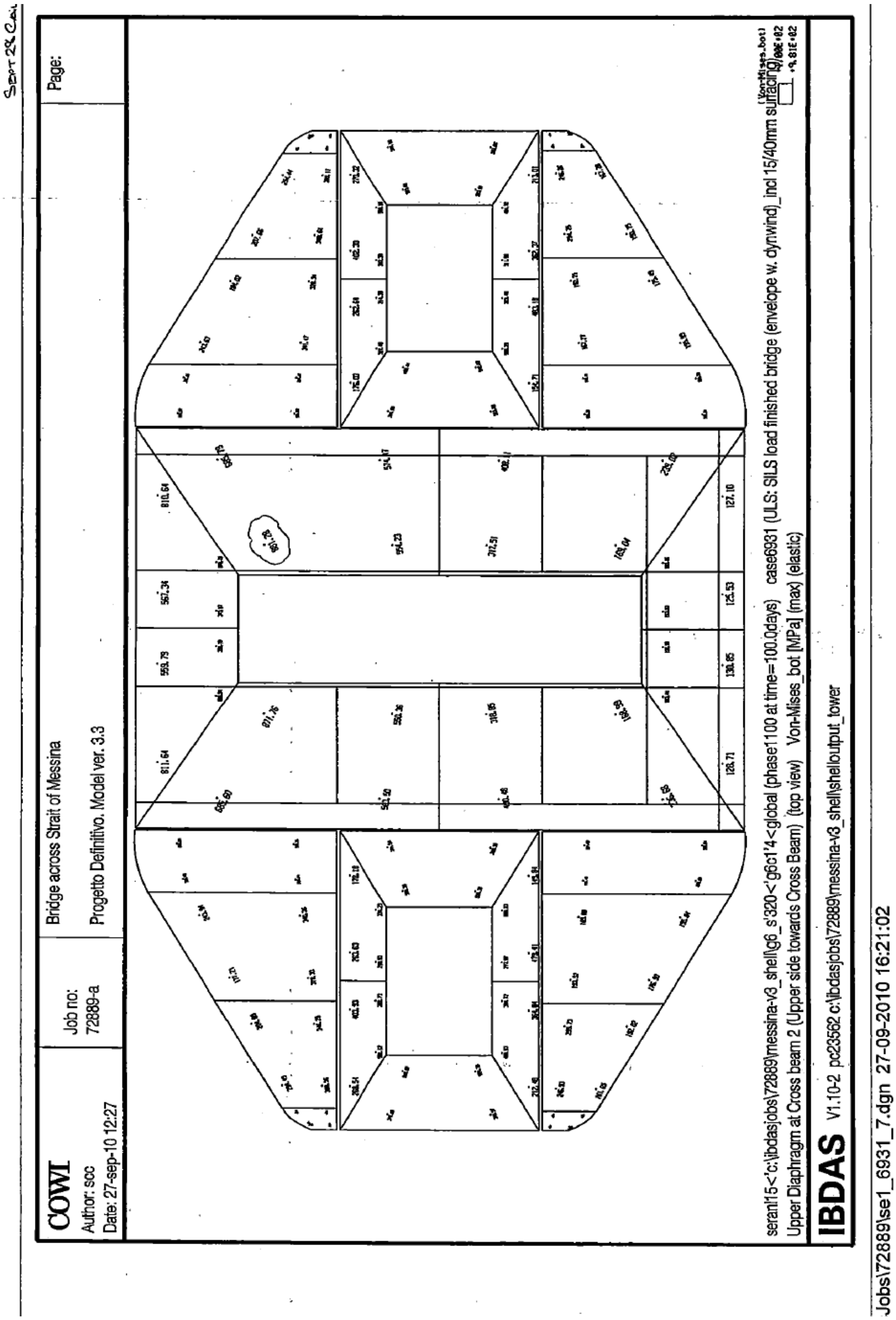
SEPT 23 Calculations(cont) page

COWI Author: scc Date: 30-sep-10 10:55	Job no: 72889-a	Bridge across Strait of Messina Progetto Definitivo. Model ver. 3.3	Page:

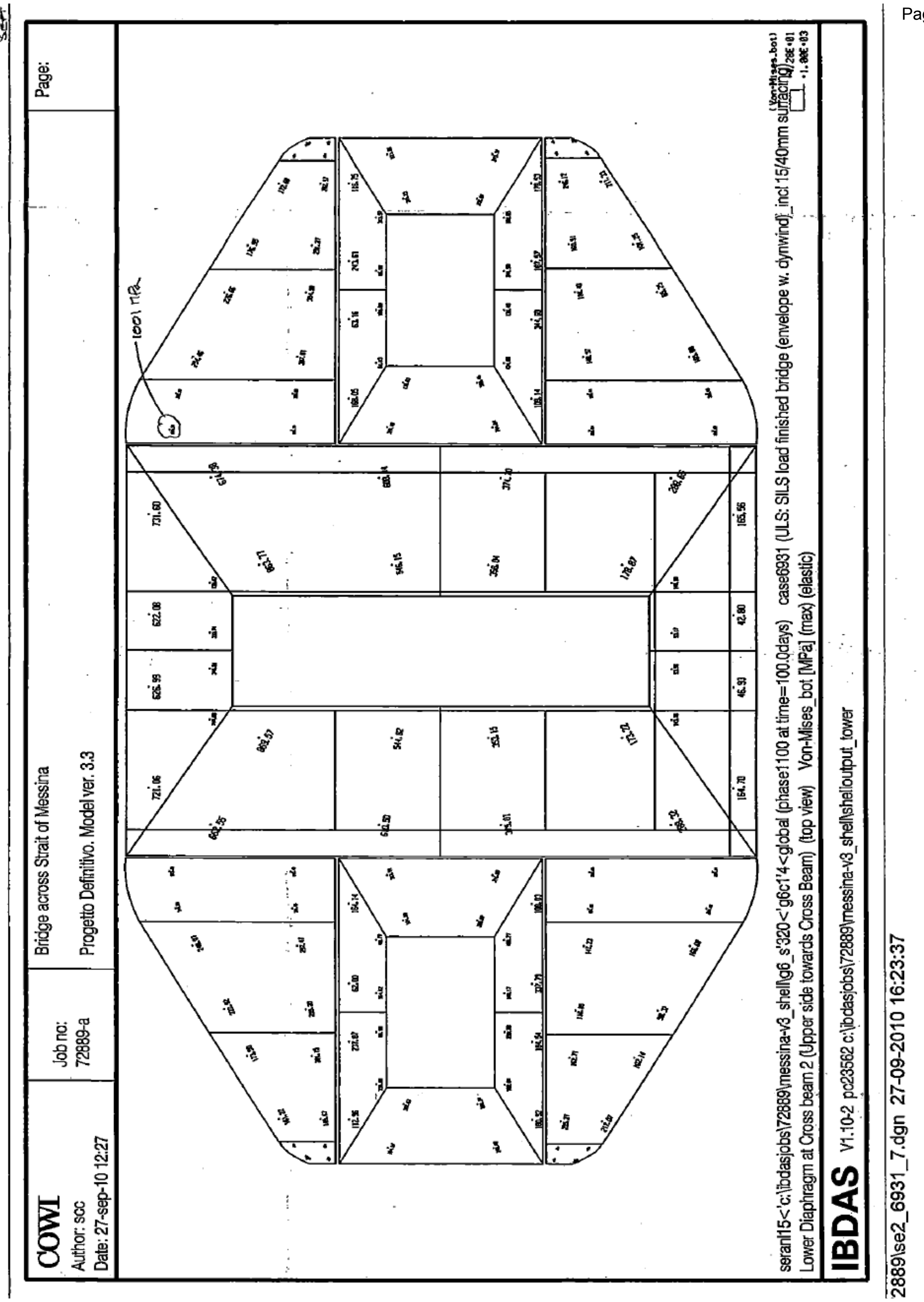


seranl19<'c:\ibdasjobs\72889\messina-v3_shell\g6_s'320<'g6c1'4<global (phase1100 at time=100.0days) case6931 (ULS: SILS load
Wall A (Sidespan side) (left view) ssy [MPa] (min) (at Zef=0.00 in Efib=1)(elastic) -3.63E+00 -1.29E+02

IBDAS V1.10-2 pc23562 c:\ibdasjobs\72889\messina-v3_shell\shelloutput_tower


COWI
Author: scc
Date: 27-sep-10 12:27
Job no:
72889-a
Bridge across Strait of Messina
Progetto Definitivo. Model ver. 3.3
Page:
seranl15<'c:\ibdasjobs\72889\messina-v3_shell\g6_s'320<'g6c1'4<global (phase1100 at time=100.0days) case6931 (ULS: SILS load finished bridge (envelope w. dynwind)_incl 15/40mm surfacing)
Upper Diaphragm at Cross beam 2 (Upper side towards Cross Beam) (top view) Von-Mises_bot [MPa] (max) (elastic)
IBDAS V1.10-2 pc23562 c:\ibdasjobs\72889\messina-v3_shell\shell\output_tower
Jobs\72889\se1_6931_7.dgn 27-09-2010 16:21:02

Page 686

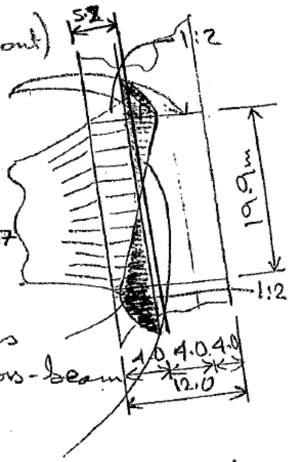

COWI
Author: scc
Date: 27-sep-10 12:27
Job no:
72889-a
Bridge across Strait of Messina
Progetto Definitivo. Model ver. 3.3
Page:
seranl15<c:\ibdasjobs\72889\messina-v3_shell\g6_s'320<'g6c14<global (phase1100 at time=100.0days) case6931 (ULS: SILS load finished bridge (envelope w. dynwind)_incl 15/40mm surfacing)
Lower Diaphragm at Cross beam 2 (Upper side towards Cross Beam) (top view) Von-Mises_bot [MPa] (max) (elastic)
IBDAS V1.10-2 pc23562 c:\ibdasjobs\72889\messina-v3_shell\shell\output_tower
2889\se2_6931_7.dgn 27-09-2010 16:23:37

SEPT 28 CALCULATIONS (cont)

BUCKLING RESISTANCE WITHOUT HORIZ STIFFENERS (cont)

HORIZONTAL STRESSES @ PLG 1st STIFFENER No horiz. stiffeners

Plate G has no horiz. stiffeners
Stresses disperse to approx zero at 45° from diaphragm
Find stress at plate G
Assume moment from cross-beam web reduced to $\frac{2}{3}\left(1-\frac{5.2}{12}\right) = 0.567$
Assume stress disperses on section at 1:2



∴ Depth of effective section
$= \frac{5.2/4}{2} + 19.9 + \frac{5.2/4}{2} = 25.1$ ~~23.9~~ m
Assume 50 mm plate

Horizontal stiffeners extend from cross-beam web stiffeners

approximation to stress distribution

∴ Inertia of section (reduced for shear lag 0.567)

Flanges → $2 \times \frac{2}{3} \times 6752 \times 40 \times 9.95^2 \times 10^6$ = 30.3 ~~35.6~~ $\times 10^{12}$ mm⁴
Stiffeners → $2 \times \frac{2}{3} \times 44.73 \times 525 \times 6 \times 9.65^2 \times 10^6$ (0.567) = 14.9 ~~17.5~~ $\times 10^{12}$
Webs → $2 \times 50 \times 25.1^3$ ~~23.9~~ $\times 10^9 / 12$ = 131.8 ~~113.8~~ $\times 10^{12}$
177 ~~166.9~~ $\times 10^{12}$ mm⁴

Assume moment decreases linearly between faces of leg
Moment resisted = $\frac{2}{3} \times 5490$ (0.567) = 3113 ~~3660~~ $\times 10^9$

∴ max stress $\frac{M_y}{I} = \frac{3113 \times 10^9 \times 9.95 \times 10^6}{177 \times 10^{12}}$ = 175 ~~218~~ MPa

HORIZONTAL STRUT BUCKLING
Check if 50 mm thickness is sufficient for 218 MPa applied

uniform compression

$\sigma_{cr} = \frac{1.0\pi^2 E}{12(1-\nu^2)}\left(\frac{50}{1600}\right)^2 = 185$ ~~330~~ MPa $\rightarrow \bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{460}{185}} = 1.577$ ~~1.394~~

∴ $\rho = \frac{\lambda_p - 0.22}{\lambda_p^2} = \frac{1.577 - 0.22}{(1.577)^2} = 0.546$ ~~0.604~~

∴ $\rho f_y / \gamma_M = \frac{0.546 \times 460}{1.0} = 251$ ~~278~~ MPa $\rightarrow \frac{D}{C} = \frac{175}{251}$ ~~218/278~~ $= 0.697$ ~~0.784~~

From page 673, coincident shear $\left(\frac{31}{50} \times 159\right)$ (99) reduces effective yield stress by von Mises (SILS t needed / t provided, page 673)

$\sigma_1 = \sqrt{f_y^2 - 3\tau^2} = \sqrt{460^2 - 3 \times 99^2} = 427$ MPa

∴ $\rho \frac{f_{y\,reduced}}{\gamma_m} = \frac{0.546 \times 427}{1.0}$ ~~0.604~~ $= 233$ ~~258~~ MPa $\rightarrow \frac{D}{C} = \frac{175}{233}$ ~~218/258~~ $= 0.751$ ~~0.844~~ ⇒ OK

SEPT 28 CALCULATIONS (cont)

BUCKLING RESISTANCE WITHOUT HORIZONTAL STIFFENERS (cont)

BUCKLING BY "DISHING"

On a square pattern of buckling
For uniaxial compression, $k = 4$

For equal compression in 2 directions $k = 2$

Simplest solution for different stresses is to sum the stresses from the two directions and proportion k from the sum

Vertical stress ~~at edge~~ at ℄ of G (at centre of 1600 ~~1200~~ mm vert stiff spacing)

$= 460 - \frac{6(8.0 - \frac{1.2}{2})}{12.0} 310 = 460 - \sout{191} = \sout{269}$ MPa (155, 305)

f_y ↗ page 670

Horizontal stress at edge of panel ~~X~~ = 175 ~~218~~ MPa ← page ~~676~~ 687

~~487 MPa~~ 480 MPa

$\therefore k_{vert} = \frac{305\ (\sout{269})}{480\ (\sout{487})} \times 4 = 2.54\ (\sout{2.21}) \rightarrow \sigma_{cr.vert} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2 = \frac{2.54\ (\sout{2.21})\ \pi^2\ 210000}{12 \times 0.91} \times \left(\frac{50}{1600\ (\sout{1200})}\right)^2 = 470\ (\sout{728})$ MPa

$k_{horiz} = \frac{175\ (\sout{218})}{480\ (\sout{487})} \times 4 = 1.46\ (\sout{1.79})$

Check according to EN1993-1-5 #10(2)

α_{ult} from von Mises stress:-

$\sigma_0^2 = \sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2 + 3\tau^2 = 305\ (\sout{269})^2 + 175\ (\sout{218})^2 - 305\ (\sout{269}) \times 175\ (\sout{218}) + 3(99)^2$ ← page 676

$\sigma_0 = \sqrt{93025\ (\sout{72361}) + 30625\ (\sout{47524}) - 53375\ (\sout{58642}) + 29403} = \sqrt{99678\ (\sout{90646})} = 316\ (\sout{301})$ MPa

$\therefore \alpha_{ult\,k} = \frac{460}{316\ (\sout{301})} = 1.457\ (\sout{1.528})$

$\alpha_{cr} = \frac{470\ (\sout{728})}{305\ (\sout{269})} = 1.541\ (\sout{2.71})$

$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult}}{\alpha_{cr}}} = \sqrt{\frac{1.457\ (\sout{1.528})}{1.541\ (\sout{2.71})}} = 0.972\ (\sout{0.751})$

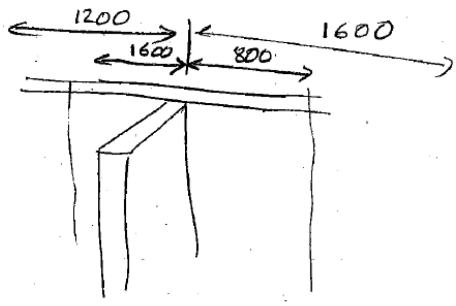
$\therefore \rho = \frac{\bar{\lambda}_p - 0.22}{\bar{\lambda}_p^2}$ ← uniform compression $= \frac{0.972\ (\sout{0.751}) - 0.22}{0.972\ (\sout{0.751})^2} = 0.796\ (\sout{0.941})$

$\therefore$ Max resistance $= \rho f_y = 0.796\ (\sout{0.941}) \times 460 = 366\ (\sout{433})$ MPa

$\therefore \left(\frac{P}{C}\right) = \frac{305\ (\sout{269})}{366\ (\sout{433})} = (0.833 \text{ ✓ OK})\ (\sout{0.621})$ This is the widest panel, so others will be OK.

SEPT 28 CALCULATIONS (cont)

VERTICAL STIFFENERS STABLIZING HORIZ FORCES



At ℄ of stiffener
Vertical stress = $460 - \frac{6.8}{12} \times 310 = 284$ MPa
Area of stiffener = $1400 \times 50 = 70000$ mm²
+ $600 \times 60 = 36000$
106000

∴ Axial on stiffener, $N_a = 284 \times 106000 = 30.1$ MN

Stiffener N_{bRd}; $e_{c\,max} = 214.6$
$i = 184.4$

$$\alpha_e = \alpha + \frac{0.09}{i/e} = 0.49 + \frac{0.09}{(184.4/214.6)} = 0.595$$

$$\lambda = \frac{L_{cr}}{i} = \frac{3333}{184.4} = 18.07$$
$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = \pi\sqrt{\frac{210000}{460}} = 67.1$$
$$\bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{18.07}{67.1} = 0.269$$

∴ $\chi = 0.958$

$$\frac{\chi f_y}{\gamma_{M1}} = \frac{0.958 \times 460}{1.0} = 440 \text{ MPa} \rightarrow N_{bRd} = 46.7 \text{ MN}$$ page 691

(Gamma M1 — S1.3)

$$N_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 \times 210 \times 10^3 \times 3.61 \times 10^9}{(3.333)^2 \times 10^6} = 673 \text{ MN}$$ (page 690)

Bending stress in first yield model = $\frac{(1-\chi) f_y}{\gamma_M} = \frac{(1-0.958)460}{1.0} = 19.3$ MPa

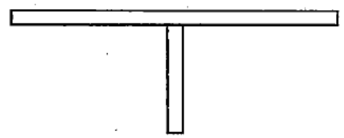
∴ Moment, $M_{Perry} = \sigma \frac{I}{y} = 19.3 \times \frac{3.61 \times 10^9}{514.6} = 135.3$ MN-mm

∴ $e_{Perry} = \frac{M}{N_{bRd}} = \frac{135.3}{46.7} = 2.90$ mm

∴ $e_{0\,Perry} = e_{Perry} \times (1 - N/N_{cr}) = 2.90\left(1 - \frac{46.7}{673}\right) = 2.70$ mm

$\frac{N_L}{b}$ = Plate thickness × horizontal stress = 50×175 MPa = 8750 N/mm (page 687)

I properties



15*t*epsilon = 610.2
2*15*t*epsilon = 1220.4

					Aiycg		Aiy2	
Item	Bi	Di	Ai	ycg	Ai*ycg	ycgNA	Ai*ycgNA^2	Iself
Skin plate	1400.0	50.0	70000.0	25.00	1,750,000	-110.4	852,821,289	14,583,333
Outstand flange	60.0	600.0	36000.0	350.00	12,600,000	214.6	1,658,263,617	1,080,000,000
		650.0	106000.0		14,350,000		2,511,084,906	1,094,583,333
			yNA =	135.38				2,511,084,906
			yo =	514.62			Igross =	3,605,668,239
			W top	26,634,204	e_c1	110.4	i =	184.4
			W bottom	7,006,431	e_c2	214.6		
					e_c max	214.6		

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	gammaM1	plate thickness t	A/mm	alpha	Length actual	eff length factor	Lcr	iz	lambdaz Lcr/iz	E	fy	lambda1 pSQRT(E/fy)	lambdabarz	phiz	chiz	allowable stress	NbRd flux (N/mm)	NplRd
ate	1.1	60	60	0.49	1,600	1.00	1,600	17.3	92.4	210000	460	67.1	1.376	1.735	0.358	150	8,987	
ate	1.1	60	60	0.49	1,500	1.00	1,500	17.3	86.6	210000	460	67.1	1.290	1.599	0.393	164	9,860	
ate	1.1	60	60	0.49	1,325	1.00	1,325	17.3	76.5	210000	460	67.1	1.140	1.380	0.464	194	11,631	
ate	1.1	60	60	0.49	1,200	1.00	1,200	17.3	69.3	210000	460	67.1	1.032	1.237	0.522	218	13,085	
ate	1.1	60	60	0.49	1,126	1.00	1,126	17.3	65.0	210000	460	67.1	0.968	1.157	0.558	234	14,011	
ate	1.1	55	55	0.49	1,600	1.00	1,600	15.9	100.8	210000	460	67.1	1.501	1.946	0.314	131	7,225	
ate	1.1	55	55	0.49	1,500	1.00	1,500	15.9	94.5	210000	460	67.1	1.407	1.786	0.346	145	7,969	
ate	1.1	55	55	0.49	1,325	1.00	1,325	15.9	83.5	210000	460	67.1	1.243	1.528	0.414	173	9,514	
ate	1.1	55	55	0.49	1,200	1.00	1,200	15.9	75.6	210000	460	67.1	1.126	1.361	0.471	197	10,824	
ate	1.1	55	55	0.49	1,126	1.00	1,126	15.9	70.9	210000	460	67.1	1.057	1.268	0.508	212	11,681	
ate	1.1	50	50	0.49	1,600	1.00	1,600	14.4	110.9	210000	460	67.1	1.651	2.219	0.270	113	5,649	
ate	1.1	50	50	0.49	1,500	1.00	1,500	14.4	103.9	210000	460	67.1	1.548	2.029	0.299	125	6,260	
ate	1.1	50	50	0.49	1,325	1.00	1,325	14.4	91.8	210000	460	67.1	1.368	1.721	0.361	151	7,558	
ate	1.1	50	50	0.49	1,200	1.00	1,200	14.4	83.1	210000	460	67.1	1.239	1.521	0.416	174	8,694	
ate, need 50mm plate minimum to carry shear in panel + vertical load																		
																	NbRd (N)	
rt stiff on G	1.1		106,000	0.49	3,333	1.00	3,333	184.4	18.1	210000	460	67.1	0.269	0.553	0.965	403	42,767,520	
rt stiff on G	1.1		#REF!	0.49	3,333	1.00	3,333	#REF!	#REF!	210000	460	67.1	#REF!	#####	#REF!	#REF!	#REF!	
stiffener	1.0	50	106,000	0.595	3,333	1.00	3,333	184.4	18.1	210000	460	67.1	0.269	0.557	0.958	440	46,691,868	

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SEPT 28 CALCULATIONS (cont)

VERTICAL STIFFENERS STABILIZING HORIZ FORCES (cont)

Stabilizing reaction = Destabilizing load

$$\therefore EI\left(\frac{\pi}{b}\right)^4 w_1 = \left[\frac{N_L}{b}\left(\frac{w_1+w_0+e_1}{a_1}\right)+\frac{N_L}{b}\left(\frac{w_1+w_0+e_2}{a_2}\right)+N_a(w_1+w_{0c})\left(\frac{\pi^2}{b^2}\right)+F_d\right]$$

Fd is the externally applied load = 0 for these cases

$$\therefore EI\left(\frac{\pi}{b}\right)^4 w_1 = \frac{N_L}{b}\left(\frac{w_1}{a_1}\right)+\frac{N_L}{b}\left(\frac{w_0+e_1}{a_1}\right)+\frac{N_L}{b}\left(\frac{w_1}{a_2}\right)+\frac{N_L}{b}\left(\frac{w_0+e_2}{a_2}\right)+N_a w_1\left(\frac{\pi^2}{b^2}\right)+N_a w_{0c}\left(\frac{\pi^2}{b^2}\right)$$

$$\therefore EI\left(\frac{\pi}{b}\right)^4 w_1 - \frac{N_L}{b}\left(\frac{w_1}{a_1}\right)-\frac{N_L}{b}\left(\frac{w_1}{a_2}\right)-N_a w_1\left(\frac{\pi^2}{b^2}\right) = \frac{N_L}{b}\left[\left(\frac{w_0+e_1}{a_1}\right)+\left(\frac{w_0+e_2}{a_2}\right)\right]+N_a w_{0c}\left(\frac{\pi^2}{b^2}\right)$$

$$\therefore \left[EI\left(\frac{\pi}{b}\right)^4 - \frac{N_L}{ba_1}-\frac{N_L}{ba_2}-N_a\left(\frac{\pi}{b}\right)^2\right]w_1 = \frac{N_L}{b}\left[\left(\frac{w_0+e_1}{a_1}\right)+\left(\frac{w_0+e_2}{a_2}\right)\right]+N_a w_{0c}\left(\frac{\pi^2}{b^2}\right)$$

$W_0 = \frac{s}{300} = \frac{1200}{300} = 4.0mm, e_1 = e_2 = 0$

$$\therefore [P-S]\, w_1 = 8750\left[\frac{4}{1200}+\frac{4}{1600}\right]+30.1\times10^{6}\times 11\times\left(\frac{\pi}{3333}\right)^2$$

$$= 51.0 + 294 = 345$$

$EI\left(\frac{\pi}{b}\right)^4 = 210\times10^3\times3.61\times10^9\times\left(\frac{\pi}{3.333}\right)^4\times10^{-12} = 598$

$\frac{N_L}{ba_1} = \frac{8750}{1200} = -7$

$\frac{N_L}{ba_2} = \frac{8750}{1600} = -5$

$N_a\left(\frac{\pi}{b}\right)^2 = 30.1\times10^6\times\left(\frac{\pi}{3333}\right)^2\times10^{-6} = -27$

557

$\therefore 557 w_1 = 345 \rightarrow w_1 = \frac{345}{557} = 0.619$

$$E\left(\frac{\pi}{b}\right)^4 I_{min}\, w_1 = 8750\left(\frac{0.62+4+0}{1200}\right)+8750\left(\frac{0.62+4}{1600}\right)+30.1\times10^6\left(0.62+11.1\right)\left(\frac{\pi}{3.333}\right)^2\times10^{-6}$$

(11.72)

$$= 33.7 + 25.3 + 313.4 = 372$$

$$E\left(\frac{\pi}{b}\right)^4 w_1 = 210\times10^3\left(\frac{\pi}{3.333}\right)^4\times10^{-12}\times0.62 = 102.8\times10^{-9}$$

$$\therefore I_{min} = \frac{372}{102.8\times10^{-9}} = \frac{372}{102.8}\times10^9 = 3.62\times10^9$$

$M = 419$

Page 693

	A	B	C	D	E
1			G inner	G outer	Outer H inner
2	from outer face		6800	5200	2500
3	Modulus	Es	210,000	210,000	210,000
4	Poisson's ratio	nu	0.30	0.30	0.30
5	Shear modulus	G	80,769	80,769	80,769
6	f_y		460	460	460
7	GammaM0	GammaM0	1.0	1.0	1.0
8	GammaM1	GammaM	1.0	1.0	1.0
9	fy/GammaM1		460	460	460
10	axial stress along stiffener	s_a	284	326	395
11	available bending	s_bmax	176	134	65
12					
13	transverse stiffener span	b	3333	3333	3333
14	transverse stiffener spacing	a_1	1200	1200	1200
15	transverse stiffener spacing	a_2	1600	1600	1600
16	plate CL eccy 1	e_1	0	0	0
17	plate CL eccy 2	e_2	0	0	0
18					
19	number of longitudinal stiffeners	nls	0	0	0
20	thickness of longitudinal stiffeners	tls	0	0	0
21	breadth of longitudinal stiffeners	bls	0	0	0
22					
23	skin breadth=(a_1+a_2)/2	bskf	1400	1400	1400
24	skin thickness (plate H/G/H = skin)	tskf	50	50	50
25	transverse stiffener web breadth	bstw	450	475	550
26	transverse stiffener web thickness	tstw	45	48	55
27	transverse stiffener flange breadth	bstf	0	0	0
28	transverse stiffener flange thickness	tstf	0	0	0
29	transverse stiffener depth	Dst	500	525	600
30	Area	A_s	90250	92800	100250
31	Skin; distance of centroid from outside face of skin	yski	25	25	25
32	Web; distance of centroid from outside face of skin	ywi	275	287.5	325
33	Flange; distance of centroid from outside face of skin	yfi	500	525	600
34	Neutral axis; distance of centroid from outside face of skin	yNA	81.09	89.49	115.52
35	Skin; distance of centroid from NA	yNAski	56.1	64.5	90.5
36	Web; distance of centroid from NA	yNAwi	-193.9	-198.0	-209.5
37	Flange; distance of centroid from NA	yNAfi	-418.9	-435.5	-484.5
38	I provided	I	1,337,950,283	1,628,339,529	2,678,132,923
39					
40	centroid of skin from NA	ec_sk	56.09	64.49	90.52
41	centroid of stiffener from NA	ec_st	193.91	198.01	209.48
42	e centroid	e_c	193.9	198.0	209.5
43					
44	y0 = e_max	y	418.9	435.5	484.5
45	radius of gyration	is	121.8	132.5	163.4
46	alpha_s	apha_s	0.49	0.49	0.49
47	alpha_e	alpha_e	0.633	0.625	0.605
48	lambda_s = b/is		27.4	25.2	20.4
49	lambda1_s =		67.1	67.1	67.1
50	lambdab_s		0.408	0.375	0.304
51	phi_s=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)		0.649	0.625	0.578
52	chi_s =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1		0.867	0.889	0.936
53	resistance stress =chi_s*f_y/GammaM	sRa	398.7	409.0	430.4
54					
55	Area of plate + stiffener	A_ps = A_s	90,250	92,800	100,250
56	External compression force applied to transverse stiffener = s_a*A_ps	Na	25,661,083	30,221,867	39,640,521
57	axial compressive resistance of tranvserse stiffener = Nb,Rd	N	35,981,769	37,952,061	43,147,709
58	Ncr = PI()^2*Es*I/b^2	Ncr	249,625,179	303,803,924	499,666,854
59	((f_y/GammaM)-(N_bRd/A_s))		61.3	51.0	29.6
60	(1-(N/Ncr))		0.856	0.875	0.914
61	M=sb*I/y		195,819,143	190,813,231	163,619,633
62	w_0c*m=M/N		5.44	5.03	3.79
63					
64	w_0c1=(fy/gM-N/A)(I/(N*y))(1-N/Ncr)	w_0c1	4.66	4.40	3.46
65	w_0c2 = b/300	w_0c2	11.11	11.11	11.11
66	w_0c=MAX(w_0c1,w_0c2)	w_0c	11.11	11.11	7.56
67					
68	e_max	e_max	418.9	435.5	484.5
69	w_0=MIN(a_1/300, a_2/300, b/300)	w_0	4.00	4.00	4.00
70					
71	Longitudinal Stress (horizontal stress in plates H/G/H)	sL	175	134	64
72	Longitudinal force	NL	29,163,750	22,301,691	10,721,967
73	NL/b		8750	6691	3217
74	Es*I*(PI()/b)^4	f_4	221.8	269.9	443.9
75	sum of w1+w0+e1	sum1			
76	sum of w1+w0+e2	sum2			
77	Fd	Fd	0	0	0
78	(NL/b)((w0+e1)/a1+(w0+e2)/a2)+Na*w0c(PI()/b)^2+Fd	Q	304.3	337.3	284.8
79					

Page 693A

	A	B	C	D	E
80					
81	f_4-(NL/b)(1/a_1+1/a_2)-Na*(PI()/b)^2	f_5	186.22	233.30	404.02
82	w_1 = Q/f_5	w_1	1.63	1.45	0.71
83	((NL/b)((w1+w0+e1)/a1+(w1+w0+e2)/a2)+Na*(w1+w0c)(PI()/b)^2+Fd)/(Es*(PI()/b)^4*w1)	Icheck	1,337,950,283	1,628,339,529	2,678,132,923
84	Icheck/Iprovided		1.000	1.000	1.000
85	Max distributed load on stiffener = Es*I*w_1*(PI()/b)^4		362	390	313
86	Max shear force in stiffener = Es*I*w_1*(PI()/b)^3	V	384,527	414,051	332,045
87	Max bending moment in stiffener = Es*I*w_1*(PI()/b)^2	M	407,955,057	439,277,297	352,275,958
88	max bending stress = M*y/I	sbEd	127.7	117.5	63.7
89	total stress=		**412.06**	**443.15**	**459.14**
90	D/C = stress/(fy/GammaM)		0.90	0.96	1.00
91					
92					
93	Stiffener lateral buckling	Not relevant for flat stiffeners within b/t limits for Class 3			
94	Inertia of stiffener flange + half web	Istz	0		
95	Area of stiffener flange + half web	Astz	10,125.00		
96	radius of gyration	isz	0		
97	alpha_sz	apha_sz	0.49		
98	alpha_ez	alpha_ez	#DIV/0!		
99	**Unrestrained length**	**Lcrz**	**8000**		
100	lambda_sz = Lcr/isz = b/isz (because Lcr = b)		#DIV/0!		
101	lambda1_sz =	lambda1_sz	67.1		
102					
103	It	It	13,668,750		
104	Distance of flange centroid from face of skin	yf	450.0		
105	Polar moment of area of flange+web about face of skin	Ip	1,366,875,000		
106	G*It		1,104,014,423,077		
107	E*Istz*yf^2*(PI()/Lcrz)^2		0		
108	(E*Istz*yf^2*(PI()/Lcrz)^2)/Ip		0		
109	(G*It+E*Istz*yf^2*(PI()/Lcrz)^2)/Ip		808		
110	s_crz = (PI()^2*Es*Istz/b^2)/Astz	s_crz	0		
111	**FE s_cr**	**s_crzFE**			
112	elastic critical buckling stress (MUST SELECT)	s_cr	808		
113	lambdab_sz from Lcr/isz		#DIV/0!		
114	lambdab_sz from s_cr	lambdab_sz	0.755		
115	phi_sz=0.5*(1+alpha_e*(lambdab_s-0.2)+lambdab_s^2)	phi_sz	#DIV/0!		
116	chi_sz =1/(phi_s+(phi_s^2-lambdab_s^2)^0.5)<1	chi_sz	#DIV/0!		
117	resistance stress =chi_sz*f_y/GammaM	s_bRdz	#DIV/0!		
118	D/C = sbEd/s_bRdz		#DIV/0!		
119					
120	s_bRdz/s_cr		#DIV/0!		
121					
122	FE output				

Plate Sizes and Stiffener Dimensions for Sicilia Tower

CONCIO	SPESSORA LAMIERA							
	tA	tB	tC	tD	tE	tF	tG	tH
1	100	100	100	65	100	70	45	45
2	100	100	85	55	100	60	35	40
3	100	100	70	50	80	40	30	40
4	100	95	55	40	65	35	30	40
5	90	80	45	40	60	35	35	40
6	85	65	45	40	50	35	35	40
7	75	50	45	55	50	40	~~35~~ 50	~~45~~ 50
8	65	50	55	60	40	40	~~40~~ 50	~~45~~ 50
9	65	50	50	50	40	40	45	45
10	65	45	45	45	40	40	35	45
11	75	45	45	45	45	40	40	45
12	85	50	45	45	45	40	40	40
13	90	60	45	50	40	40	~~40~~ 50	~~45~~ 50
14	95	70	45	50	45	40	~~40~~ 50	~~45~~ 50
15	95	65	45	50	45	40	40	40
16	90	70	40	40	55	40	35	40
17	90	60	40	40	50	40	35	40
18	75	60	40	40	50	35	35	40
19	65	50	40	50	45	40	40	40
20	65	45	40	55	45	40	~~40~~ 50	~~45~~ 50
21	65	45	40	55	50	50	50	50

Handwritten brackets: 1 (concio 7–8), 2 (concio 13–14), 3 (concio 20–21)

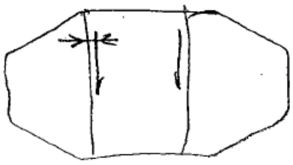
Segment 9
ULS Wind: $N_{Ed} \approx 19000$ MN ; M_{Ed}^2 15000 MN-m
SILS Wind: $N_{Ed} \approx 17700$ MN ; $M_{Ed} \approx 1839$ MN-m

CONCIO	STIFFENER DIMENSIONS							
	tA	tB	tC	tD	tE	tF	tG	tH
1	750 x 75	700 x 70	700 x 70	650 x 65	725 x 73	600 x 60	475 x 48	475 x 48
2	750 x 75	700 x 70	675 x 68	650 x 65	725 x 73	575 x 58	450 x 45	450 x 45
3	750 x 75	700 x 70	675 x 68	625 x 63	725 x 73	550 x 55	450 x 45	450 x 45
4	750 x 75	700 x 70	675 x 68	600 x 60	700 x 70	525 x 53	450 x 45	450 x 45
5	750 x 75	675 x 68	625 x 63	600 x 60	675 x 68	500 x 50	450 x 45	450 x 45
6	750 x 75	675 x 68	625 x 63	600 x 60	650 x 65	500 x 50	450 x 45	475 x 48
7	750 x 75	675 x 68	625 x 63	625 x 63	625 x 63	475 x 48	475 x 48	~~500 x 50~~ 550x55
8	750 x 75	675 x 68	650 x 65	650 x 65	625 x 63	475 x 48	475 x 48	~~500 x 50~~ 550x55
9	750 x 75	650 x 65	625 x 63	625 x 63	600 x 60	475 x 48	475 x 48	525 x 53
10	750 x 75	650 x 65	600 x 60	625 x 63	575 x 58	450 x 45	450 x 45	500 x 50
11	750 x 75	650 x 65	600 x 60	625 x 63	575 x 58	450 x 45	450 x 45	500 x 50
12	750 x 75	650 x 65	600 x 60	600 x 60	575 x 58	450 x 45	450 x 45	500 x 50
13	750 x 75	675 x 68	600 x 60	575 x 58	550 x 55	450 x 45	~~450 x 45~~	~~475 x 48~~ 550x55
14	750 x 75	675 x 68	575 x 58	575 x 58	550 x 55	450 x 45	~~450 x 45~~	~~475 x 48~~ 550x55
15	750 x 75	675 x 68	575 x 58	550 x 55	550 x 55	450 x 45	450 x 45	475 x 48
16	750 x 75	675 x 68	550 x 55	550 x 55	550 x 55	450 x 45	450 x 45	450 x 45
17	750 x 75	675 x 68	550 x 55	550 x 55	550 x 55	450 x 45	450 x 45	425 x 43
18	750 x 75	675 x 68	575 x 58	550 x 55	550 x 55	475 x 48	450 x 45	475 x 48
19	750 x 75	650 x 65	575 x 58	575 x 58	550 x 55	500 x 50	475 x 48	500 x 50
20	750 x 75	650 x 65	575 x 58	600 x 60	575 x 58	500 x 50	500 x 50	~~500 x 50~~ 550x55
21	750 x 75	650 x 65	575 x 58	600 x 60	600 x 60	550 x 55	550 x 55	550 x 55

Handwritten annotations: Beam 1 (concio 7–8); X Beam 2 (concio 13–14); 475x48; 475x48; 550x55 (concio 20–21)

DIAPHRAGMS

WELDS FROM CENTAL DIAPHRAGM TO H/G/H
Using IBDAS model 3.3a output:-
Find max shear in central diaphragm = SSY
+ " ⊥ direct stress —"— = SSS



adjacent to plates H/G/H
Upper diaphragm

	244.6	SSY	62.7
SSS =	267.5		16.5
	278.0		32.6
	286.4		137.7

SSS = 54.1 SSY 309.4 ← worst case because shear is greater than any direct stresses

Lower diaphragm

SSS = 361 SSY 328.8 case (beam)
SSS = 142.8 SSY = 339.5

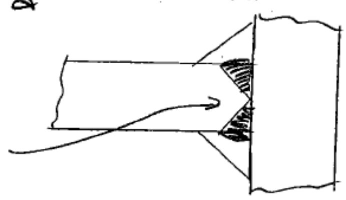
Worst case ×20mm plate → 361×20 =

From shear alone from page 611, shear each edge of diaphragm
$= \frac{1}{2} \times \frac{M}{height} = \frac{1}{2}\frac{3625}{19.9} = 91.1\ MN$

Average Shear $= \frac{91.9 \times 10^3}{12000} = 7.66\ kN/mm$

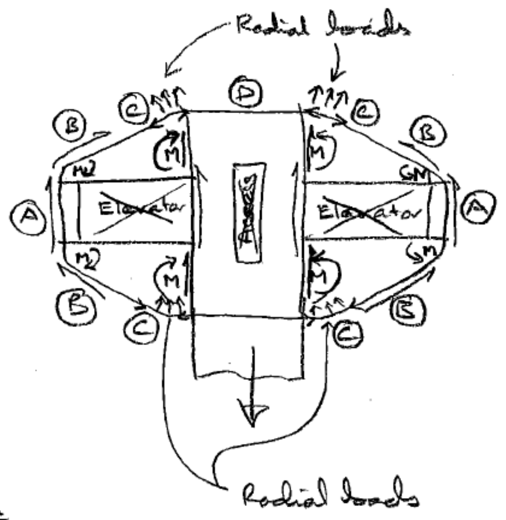
$f_{vwd} = \frac{f_u/\sqrt{3}}{\beta\ \gamma_{M2}} = \frac{540/\sqrt{3}}{1.0 \times 1.25} = 249\ MPa$ → Min Σ throat $= \frac{7.66 \times 10^3}{249} = 30.1mm$

⇒ 2 @ a = 16
⇒ 2 @ leg = $16\sqrt{2}$ = 23mm = big!



⇒ Better to use complete penetration butt because total weld is less than 23mm legs.

⇒ This must be sufficient because it is as strong as the plate.



OVERALL BEHAVIOUR

1) The inner diaphragm has a hole for access, but this is well framed and the loading is approximately symmetric about the centre-line of the hole.

2) The outer diaphragms have holes for the elevators and asymmetric load because of the high shear being transmitted to plates A&B. These shears produces
   (i) overall bending
   (ii) local (Vierendeel) bending "across" the hole

3) The curved corners, especially in plates C, give rise to radial forces in the skin from horizontal forces (produced by overall and local bending in the diaphragm). The radial forces have two big effects

   (i) The radial force causes radial displacements that reduce the effective width of the skin acting as a flange resisting bending in the diaphragm

   (ii) When the skin carries horizontal compression forces from diaphragm bending, the radial forces apply big tensile forces to the welds between the skin and the diaphragm.

WELDS – DIAPHRAGM TO VERTICAL PLATES

FORCES

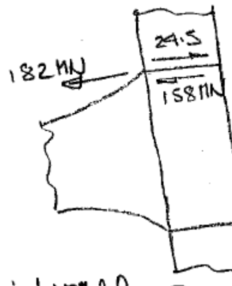
From page 602, cross beam 2 end moment 3550 MN·m with shear = 140 MN

Use original design values $M = 3625$ & $V = 145$ MN
so that early calculations can be used for direct comparison.

From page 671 leg shear above connection $= \frac{0.8M}{118.6} = \frac{0.8 \times 3625}{118.6} = 24.5$ MN

Flange force < $\frac{M}{\text{height}} = \frac{3625}{19.9} =$ 182 MN
− 24.5
= 158 MN

because some carried in web
Assume 80% from flange & 20% from web
→ $0.80 \times 158 = 126$ MN

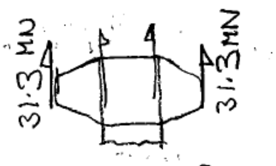


SHEAR FROM OVERALL BENDING BEHAVIOUR

From page 673 shear carried in plates ABC
= 0.5 × total, shared as page 682 10% above, 80% at diaph, 10% below

$\frac{1}{2} \times 0.5 \times 0.8 \times 126 = 25.2$ MN on plate B/A/B from within connection

$+ \frac{1}{2} \times 0.5 \times 24.5 = 6.1$ MN on plate B/A/B from above connection
∴ $V_{Ed} = 31.3$ MN



Shear in welds is worse for larger effective flange
∴ Use 50 t and 10 t effective (curvature reduces effectiveness)
Inertia of overall diaphragm

∴ $I = 2 \times 1000 \times 50 \times 6000^2 = 3.60 \times 10^{12}$ mm^4
$2 \times 4000 \times 60 \times 4000^2 = 7.68 \times 10^{12}$
$2 \times 60 \times (4000)^3/12 = 0.64 \times 10^{12}$
$= 11.92 \times 10^{12}$

Sicilia CB2 tC = 50 t
Calabria tC = 45/50 t

Shear/mm $= \frac{V_{Ed} S}{I} = \frac{31.3 \times 10^6 \times (1000 \times 50 \times 6000)}{11.92 \times 10^{12}} = 788$ N/mm

SHEAR FROM LOCAL (VIERENDEEL) BENDING BEHAVIOUR

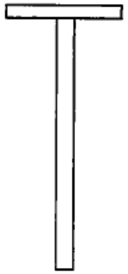
Σ Shear = 31.3 MN/per outer diaphragm
∴ Shear/triangular diaphragm $= \frac{31.3}{2} = 15.7$ MN

At deep end Shear/mm $= \frac{V_{Ed} S}{I} = \frac{15.7 \times 10^6 \times (1000 \times 50 \times 1676)}{490 \times 10^9} = 2685$ N/mm
(page 2213)

TOTAL LONGITUDINAL SHEAR

Shear from overall bending = 788 N/mm
— " — local — " — = 2685
= 3473 N/mm

I properties
For shear in welds calculation



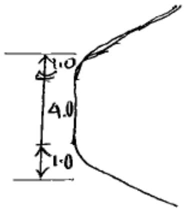
15*t*epsilon = 610.2
2*15*t*epsilon = 1220.4

Item	Bi	Di	Ai	ycg	Aiycg Ai*ycg	ycgNA	Aiy2 Ai*ycgNA^2	Iself
Skin plate	1000.0	50.0	50000.0	25.00	1,250,000	-1675.9	140,425,683,710	10,416,667
Outstand flange	60.0	4000.0	240000.0	2050.00	492,000,000	349.1	29,255,350,773	320,000,000,000
		4050.0	290000.0		493,250,000		169,681,034,483	320,010,416,667
			yNA =	1700.86				169,681,034,483
			yo =	2349.14			Igross =	489,691,451,149
			W top	287,907,797			i =	1,299.5
			W bottom	208,455,810				

E:\1913\cmk_1913\beam-leg_connection\EN_strut_buckling_2010m10d26.xls10/26/201010:03 AM

WELDS – DIAPHRAGM TO VERTICAL PLATES

PL A TOTAL SHEAR



Assume shear distributed onto plate A + B
Take effective depth = 1m of plate B + 4m of A + 1m PL B

∴ Average shear load/mm = $\frac{31.3 \times 10^6}{6000}$ = (5220 N/mm)

WELD SIZES

$f_{vwd} = \frac{f_u/\sqrt{3}}{\beta \gamma_{M2}} = \frac{540/\sqrt{3}}{1.0 \times 1.25} = 249$ MPa

PL A

∴ For plate A, min throat = $\frac{5220}{249}$ = 21.0 mm ⇒ ~~2@ throat = 12mm ⇒ leg = $12\sqrt{2}$ = 17mm~~

⇒ superseded by page 2215 ⇒ 13.1mm & 18.5mm

PL C

Longitudinal shear = 3473 N/mm from page 2212

There is an additional radial load from the curvature of the flange.

Assume flange = 50×1000 effective width @ $\frac{f_y}{\gamma_{M0}} = \frac{460}{1.05} = 438$ MPa

∴ Flange force, F = 50×1000×438 = 21.9 MN

∴ Radial load = $\frac{F}{Radius} = \frac{21.9 \times 10^6}{2000}$ = 10950 N/mm

Assume 2@20 throats ⇒ $\tau_{\parallel} = \frac{3473}{2 \times 20} = 87$ MPa

$\sigma_{\perp} = \tau_{\perp} = \frac{10950}{2 \times 20} \times \frac{1}{\sqrt{2}} = 194$ MPa

$\left[\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)\right]^{0.5} = \left[194^2 + 3(194^2 + 87^2)\right]^{0.5} = \left[37636 + 3(37636 + 7569)\right]^{0.5}$

$\frac{f_u}{\beta \gamma_{M2}} = \frac{460}{1.0 \times 1.25} = 368$ → $\frac{D}{C} = \frac{416}{368} = 1.130$ on 2@20mm throats

= 416 MPa

⇒ Use 2 @ 1.13×20 = 2@ 23mm

WELDS – DIAPHRAGM TO VERTICAL PLATES (cont)

From IBDAS shell elements in Model 3.3a

Shear stresses by plate A

Lower diaphragm shear → 92.8 × 20 pl = 1856 N/mm

Upper diaphragm shear → 217.7 × 20 pl = 4354 N/mm

These increase for increase of thickness due to higher stiffness.

From page 682, increase expected to be × 1.33

4354 × 1.33 = (5791 N/mm) compared with 5220 on page 2214

Average from shell element = 1/4 { 217.7 + 206.2 + 214.8 + 214.8 }

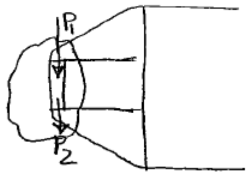
= 1/4 × 853.5 = 213.4

213.4 × 20 pl × 1.33 = (5676 N/mm) v 5220 page 2214

5676 / 5220 = 1.09 ⇒ reasonably close.

pl A: Throat required = 1.09 × 12mm (page 2219) ⇒ (a = 13.1 mm → leg = 18.5mm)

DIAPHRAGM WEB AT PLATE A



From page 2215

Shear/mm along plate A = 5791 N/mm max

= 5676 N/mm mean

For shear

∴ Minimum plate thickness for shear $= \dfrac{V_{Ed}/mm}{\left(\dfrac{f_y/\sqrt{3}}{\gamma_{M0}}\right)} = \dfrac{5791}{\left(\dfrac{460/\sqrt{3}}{1.05}\right)} = \dfrac{5791}{253} = 22.9\,mm$

For direct stress

Maximum axial load, sum of $P_1 + P_2$, = length × shear
$= 4000 \times 5676 = 22.7\,MN$

Assume 60% : 40% distribution between P_1 & P_2

$\therefore \hat{P} = 0.6 \times 22.7 = 13.6\,MN$

$\therefore$ Min tensile area $= \dfrac{\hat{P}}{\left(\dfrac{f_y}{\gamma_{M0}}\right)} = \dfrac{13.6 \times 10^6}{\left(\dfrac{460}{1.05}\right)} = \dfrac{13.6 \times 10^6}{438} = 31050\,mm^2$

From page 2217 and 2218, web depth = 1370mm & t = 20mm

$\dfrac{31050}{1370} = 22.7\,mm$

25 t might be OK but leaves little margin for approximations.

⇒ Change to 30 t webs

Z GRADE PLATE REQUIREMENTS

Refer to EN1993-1-10 Table 3.2 and EN1993-1-1 Table 3.2

PA; 30 PL web → PL A

Z_a, weld depth ≤ 15mm → 10 < a_{eff} ≤ 20 → a = 14mm → $Z_a = 6$

Z_b, shape & position — partial & full pen $Z_b = 5$

Z_c, 70 < s (plate A up to 80 thick) $Z_c = 15$

Z_d, $Z_d = 0$

26

Z_e for preheat ≥ 100°C − 8

18 < 20 ⇒ Z15

⇒ Z15 with preheat ≥ 100°C

PB; 30 PL web → PL B

Z_a =		6
Z_b =		5
Z_c =	65 PL	12
Z_d		3
Preheat < 100°C		26
— " — ≥ 100°C		− 8
		18 < 20 ⇒ Z15 if preheat ≥ 100°C

PC; 60 PL diaphragm, full penetration butt

Z_a = weld height = 30mm → a = 21mm → $Z_a = 9$

Z_b = 5

Z_c 70 PL = 15

Z_d = 0

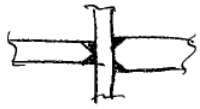
29

Preheat ≥ 100°C − 8

21 > 20 ⇒ Z25 even with preheat ≥ 100°C

Z GRADE PLATE REQUIREMENTS (cont)

PL H; 60 PL diaphm + 40 PL diaphm



Z_a = weld depth = $\frac{40}{2}$ = 20mm ⇒ a = 14 → Z_a = 6

Z_b = → Z_b = 5

Z_c 55 PL → Z_c = 12

Z_d = heavily restrained → Z_d = 5

Z_e preheat ≥ 100°C

28
− 8
20 ⇒ 20 ⇒ Z15
with preheat ≥ 100°C

PL G; 40 PL diaphm

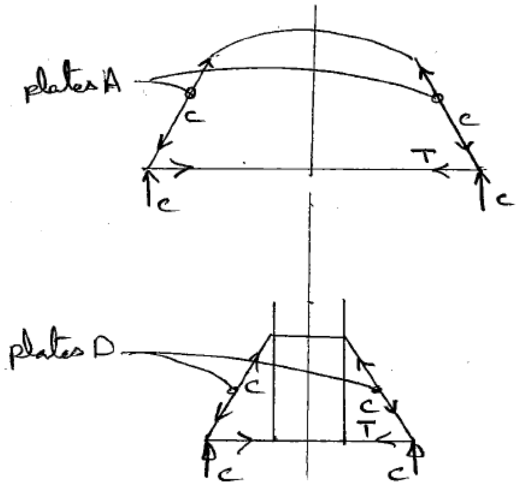
Z_a = 6
Z_b = 5
Z_c 50 PL = 10
Z_d 0
Z 21
Preheat ≥ 100°C −8
13 ⇒ Z15
with preheat ≥ 100°C

INTRODUCTION

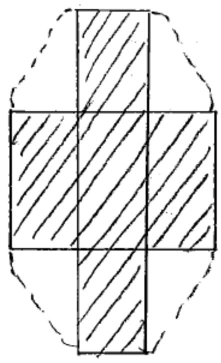
PURPOSE

This book is to size the plates forming the diaphragm at the tower top.

The diaphragm ties the slopes from plates A and D.



The diaphragm forms a cruciform because the triangular cells stop at the top of the tower and do not extend up to the cable, so are capped with inclined plates to allow rainwater to run off.



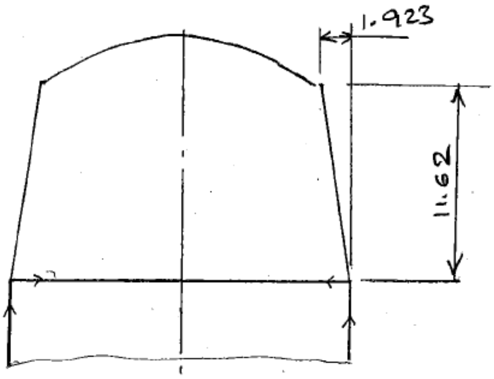
2010 Oct 14

The end-moment of the cross-beam 3 is much less than the design moment used in the calculations of cross-beam 2 and there is no opening for the elevator

=> Shear demands are much lower

FORCES FROM SADDLE

See page 1003 for geometry used.



Plates A

Horiz force $= \frac{1.923}{11.62} \times$ Vertical Force

$= 0.165 \times$ Vertical Force.

Take Vertical $=$ Area $\times \frac{f_y}{\gamma_{M0}}$

Area $= 65 \times 4000 = 260\,000\ mm^2$
$+ \quad 75 \times 750 = 56\,250$
$316\,250\ mm^2$

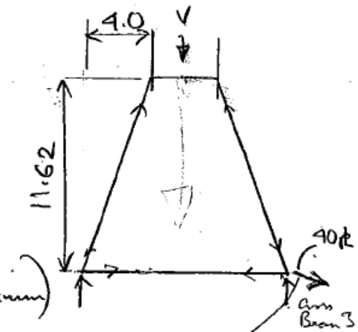
$\therefore$ Vertical force $= 316 \times 10^3 \times \frac{460}{1.05} = 138.4$ MN

$\therefore$ Horiz force $= 138.4 \times 0.165 =$ (22.8 MN)

($\therefore$ Minimum area required $= 316250 \times 0.165 = 52200\ mm^2$

On net width $= 4000 - 2 \times 600 = 2800$ mm $\rightarrow t = \frac{52200}{2800} = 18.6$ mm)
access holes
even without stresses ⊥

Plates D



Horiz force $= \frac{4.0}{11.62} \times$ Vertical Force

$= 0.344 \times$ Vertical Force

Area $= 55 \times 8000 = 440\,000\ mm^2$
$+ \ 5 \times 65 \times 650 = 211\,250$
$651\,250\ mm^2$

$\left(\frac{651250}{8000} = 81.4\ mm\right)$

Vertical force $= 651 \times 10^3 \times \frac{460}{1.05} = 285$ MN

80% of 340 MPa Wind + Catenary

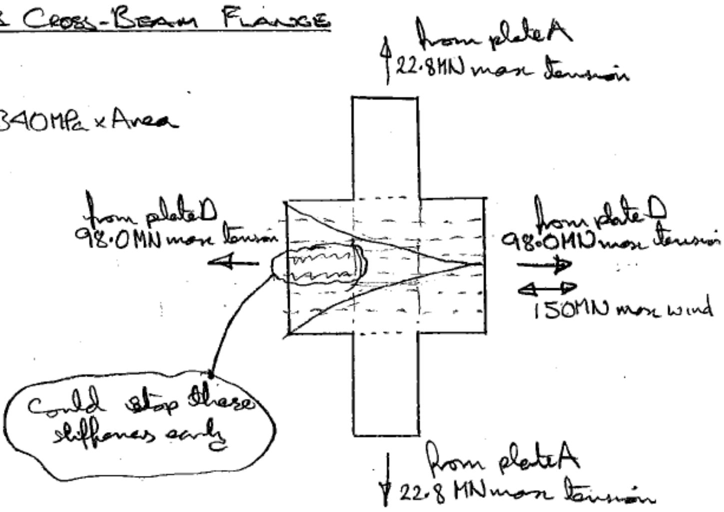
$\therefore$ Horizontal force $= 285 \times 0.344 =$ (98.0 MN)

CB3 Flange Weld 340 MPa + 0.8V

(Minimum area required $= 651\,250 \times 0.344 = 224000\ mm^2$

On net width $= 8000 - 2 \times 600 - 800 = 6000\ mm^2 \rightarrow t = \frac{224000}{6000} = 37.3$ mm)

COMBINED FORCES FROM SADDLE & CROSS-BEAM FLANGE



Wind load force

Flange force from cross beam = 340MPa × Area

Area = 40 × 8000 = 320 000 mm²
+ 6×45×450 = 121 500
441 500 mm²

∴ Force = 340 × 442×10³ = (150 MN)

Combined Wind + Vertical

∴ Max tension from Wind = 150 MN
+ 80% Max Vertical = + 0.80×98 = (78)
228 MN

(On net width (allowing for cut outs) = 8000 − 600 − 800 − 600 = 6000
∴ Min thickness (assuming no stiffeners) = $\frac{228\times10^6}{6000\times\frac{460}{1.05}}$ = 87mm plate!)

However, will continue 450 × 45 stiffeners

Find plate thickness

Area required = $\frac{228\times10^6}{(460/1.05)}$ 520000 mm²

less Area of stiffeners = 6 × 45 × 450 = −121 500
398 500 mm²

∴ Plate thickness = $\frac{398500}{6000}$ = 66.4mm ⇒ 70mm

Vertical alone

For Max vertical alone, area = $\frac{98\times10^6}{(460/1.05)}$ = 224 000mm²

less 5×45×450 stiffeners = −121 500 mm²
102 500 mm²

∴ Plate thickness = $\frac{102500}{6000}$ = 17.1 mm

COMBINED FORCES FROM SADDLE & CROSS-BEAM FLANGE (cont)

Combined Wind + Vertical (cont)

Find thickness @ 4.0m from inner plate D (assume 2/3 of 8m width effective)

∴ Wind → $(1-\frac{4.0}{12.0})150 = 100$ MN (← page 1012) from wind (reduced by shear into plates H)
2/3 flange → 80% max vertical = $\frac{2}{3}$78 = 52 MN ← page 1012

152 MN

∴ Area required = $\frac{152\times10^6}{(460/1.05)}$ = 347 000 mm²
Less Area of 4 stiffeners 4×45×450 = − 181 000
(← in the 2/3 width)
266000 mm²

From 4m to 8m from inner PL D
∴ Minimum plate thickness = $\frac{266000}{3733}$ = 71.3 mm ⇒ need full 8m width of 70 pl
width reduced by access holes = $\frac{2}{3}$×8000 − 2×800 = 3733

Find thickness @ 8.0m from inner plate D

Wind → $(1-\frac{8.0}{12.0})150$ = 50 MN from wind (reduced by shear into plates H & G)
80% max vertical = 78 MN ← page 1012

128 MN

∴ Area required = $\frac{128\times10^6}{(460/1.05)}$ = 292200 mm²
Less area of stiffeners 6×45×450 = − 121 500
170 700 mm²

From 8 & 12 m from inner PL D
∴ Minimum plate thickness = $\frac{170700}{6000}$ = 28.5 mm ⇒ 30 mm
width reduced by access holes

SHEAR FROM CROSS-BEAM FLANGE FORCE

PLATE THICKNESS REQD

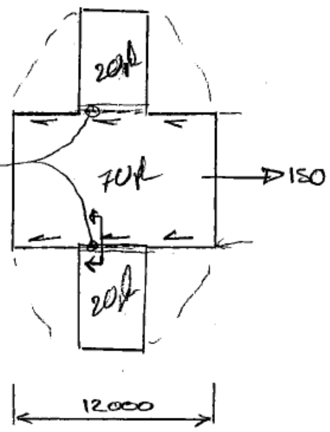
Flange force = 150 MN (page 1012)

Assuming shear distributed uniformly

∴ Load/mm, $F = \dfrac{150 \times 10^6}{2 \times 12 \times 10^3} = 6250$ N/mm

∴ Minimum thickness for shear

$$t = \frac{F}{\left(\frac{f_y/\sqrt{3}}{\gamma_{M0}}\right)} = \frac{6250}{\left(\frac{460/\sqrt{3}}{1.05}\right)} = \frac{6250}{253} = 24.7\text{ mm}$$



WELDS

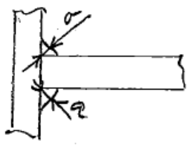
Fillets on pl H in longitudinal shear → $f_{vw,d} = \dfrac{\left(\frac{f_u}{\sqrt{3}}\right)}{\beta\,\gamma_{M2}} = \dfrac{\left(\frac{510}{\sqrt{3}}\right)}{1.0 \times 1.25} = 236$ MPa ($63 < t \leq 80$)

< shear of steel ⇒ bizarre

$\dfrac{\left(\frac{540}{\sqrt{3}}\right)}{1.0 \times 1.25} = 249$ MPa

< shear of steel ⇒ bizarre

∴

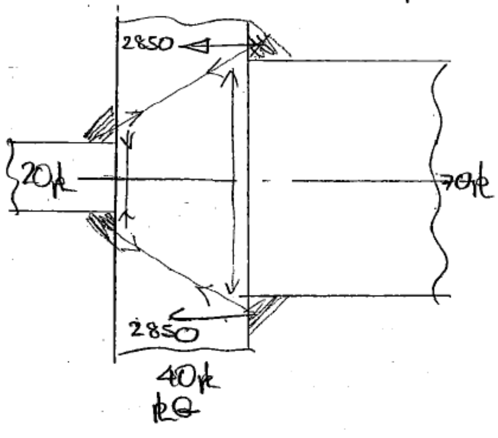


Min throat, a, required $= \dfrac{6250}{2 \times 236} = 13.2$ ⇒ 14 mm throats generally.

At plates G, transverse load/mm $= \dfrac{22.8 \times 10^6}{4000} = 5700$ N/mm (page 1011)

∴ On each of 2 fillet welds → $\dfrac{5700}{2} = 2850$ N/mm transverse

$+ \dfrac{6250}{2} = 3125$ N/mm longitudinal



∴ Direct load/mm $= \dfrac{2850}{\sqrt{2}} = 2016$ N/mm

Transverse shear $= \dfrac{2850}{\sqrt{2}} = 2016$ N/mm

longitudinal shear = 3125 N/mm²

SHEAR FROM CROSS-BEAM FLANGE FORCE (cont)

WELDS (cont)

At plate G (cont)

Resistance from #4.5.3.2 (EN1993-1-8) Directional method, $\frac{f_u}{\beta \gamma_{M2}} = \frac{510}{1.0 \times 1.25} = 408\,MPa$

The throat thickness is not defined, so assume 20 mm and ratio from that:

$\sigma_\perp = \frac{2016}{20} = 100.8\,MPa$

$\tau_\perp = \frac{2016}{20} = 100.8\,MPa$

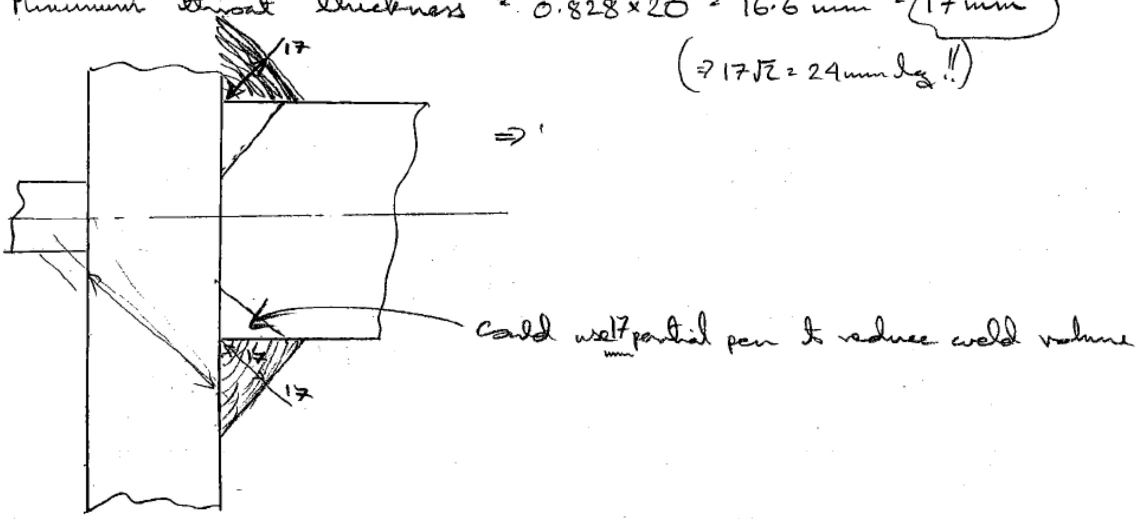
$\tau_\parallel = \frac{3125}{20} = 156.3\,MPa$

$\therefore \left[\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)\right]^{0.5} = \left[100.8^2 + 3(100.8^2 + 156.3^2)\right]^{0.5}$

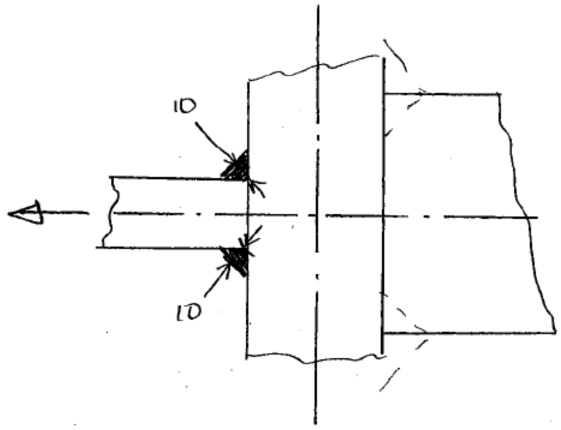
$= (10161 + 103771)^{0.5} = 338\,MPa \rightarrow \frac{D}{C}$ and $a = 20$ $= \frac{338}{408} = 0.828$

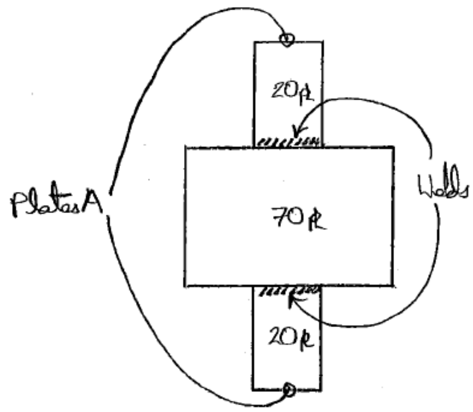
$\therefore$ Minimum throat thickness $= 0.828 \times 20 = 16.6\,mm =$ **17 mm**

($\Rightarrow 17\sqrt{2} = 24\,mm$ leg !!)



WELDS TO DIAPHRAGMS RESTRAINING PLATES A





From page 1014, direct load/mm = 2016 N/mm
& transverse shear/mm = 2016 N/mm

Try 10mm throat & scale from that

$\therefore \sigma_\perp = 201.6$ N/mm

$\tau_\perp = 201.6$ N/mm

$\tau_{\parallel} = 0$

$$\therefore \left[\sigma_\perp^2 + 3(\tau_\perp^2 + \tau_\parallel^2)\right]^{0.5} = \left[201.6^2 + 3(201.6^2)\right]^{0.5} = \left[4 \times 201.6^2\right]^{0.5} = 2 \times 201.6 = 403\,\text{MPa}$$

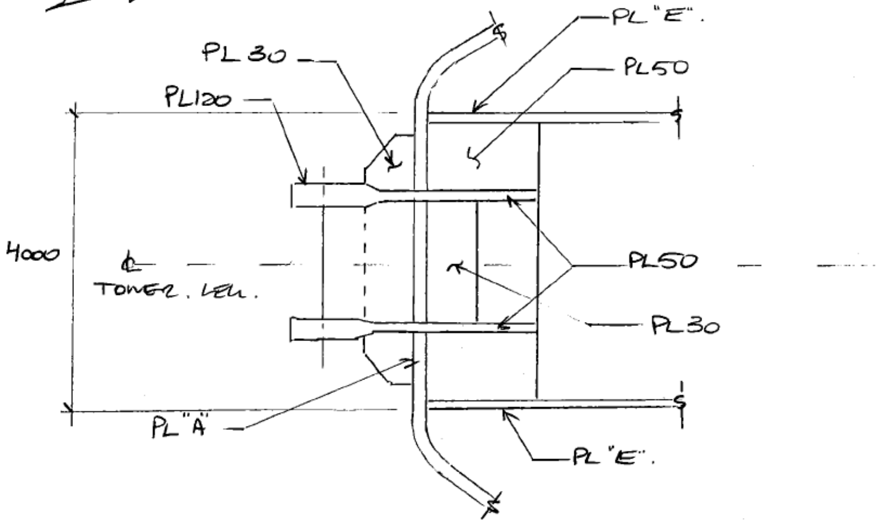
$$\frac{f_u}{\beta \gamma_{M2}} = \frac{510}{1.0 \times 1.25} = 408\,\text{MPa} \rightarrow \frac{D}{C} = \frac{403}{408} = 0.988 \Rightarrow \text{OK}$$

⇒ Use 10 mm throat welds

5.2.3 Deck Buffer Connections

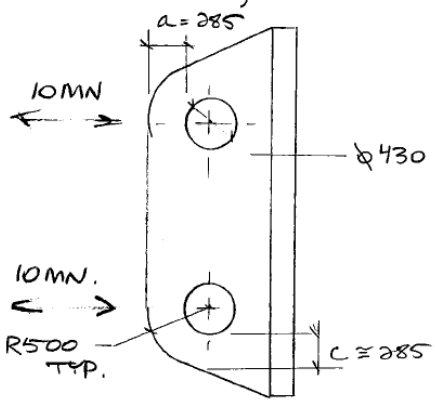
DESIGN CHECK — DECK BUFFER CONNECTION AT TOWER LEG.

→ LONGIT. BUFFER :



CHECK PIN HOLES IN PL 120:

→ EN 1993 - 1-8 § 3.13.1 TABLE 3.9



SUPERSEDED → PIN & CHANGED (2010 OCT 15)

→ $F_{Ed} = 10$ MN $d_0 = 430$ mm $a \cong c = 285$

$t = 120$ $F_y = 460$ MPa.

→ CHECK a : $\Rightarrow \frac{F_{Ed}\gamma_{M0}}{2tF_y} + \frac{2}{3}d_0 = \frac{(10\times10^6)(1.05)}{2(120)(460)} + \frac{2}{3}(430)$

$= 382$ mm $a = 285 < 382$ → NOT GOOD

→ MUST INCREASE R TO 600 MIN

→ $a = 600 - \frac{430}{2} = 385 > 380$ → O.K. ✓

→ CHECK c: → $\frac{F_{Ed}\,\gamma_{M0}}{2 t f_y} + \frac{d_0}{3} = \frac{10\times10^6 (1.05)}{2(120)(460)} + \frac{430}{3}$

$= 238$ mm < 285 mm → O.K. ✓

→ CHECK BRG RESISTANCE OF PL 120: (ASSUMED 400 φ PIN – CONSERVATIVE..)

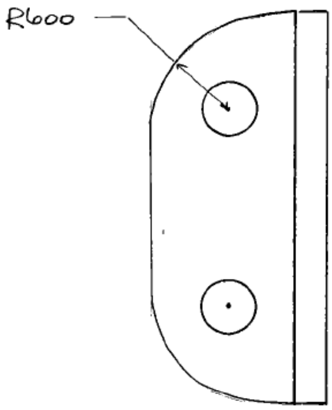
→ $F_{b,Rd} = \frac{1.5\, t\, d\, f_y}{\gamma_{M0}} = \frac{1.5(120)(400)(460)}{1.05\,(10^6)}$

$= 31.5$ MN $> F_{b,Ed} = 10$ MN → O.K. ✓

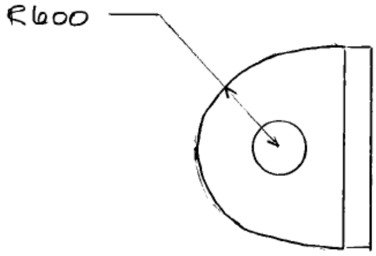
═ PIN CONNECTION AT TRANSVERSE BUFFER HAS THE SAME LOADS AND DIMENSIONS "a" AND "c"

→ MUST INCREASE R TO 600 → SEE SKETCHES BELOW:

LONGITUDINAL BUFFER:



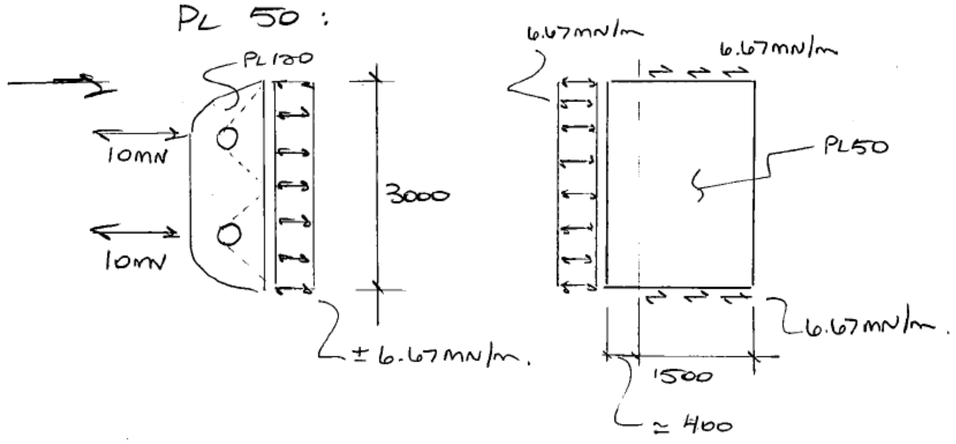
TRANSVERSE BUFFER:



S/S → PIN φ CHANGED (2010 OCT 15)

→ SEE LATER CALCS...

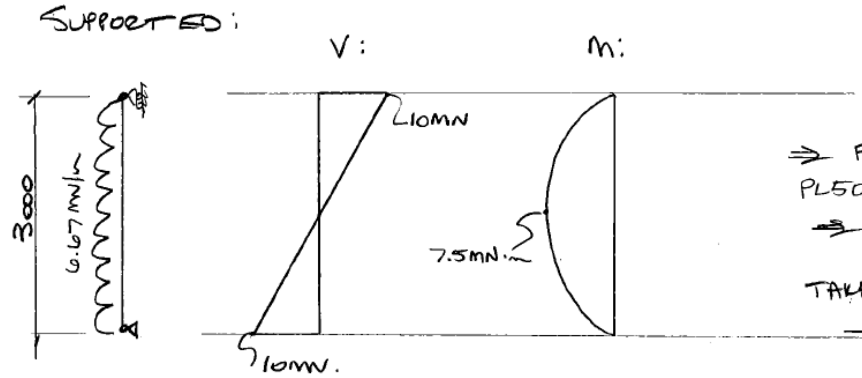
→ DISTRIBUTION OF LOAD FROM PL 120 TO PL 50 :



→ CHECK CONTACT STRESS ON 3m EDGE ;

→ $\sigma = \frac{20 \times 10^4 N}{(50)(3000)} = 133$ MPa.

→ CHECK BENDING + SHEAR — ASSUME SIMPLY SUPPORTED :



→ FOR BENDING, PL50 IS LIKE WEB:

→ $\frac{b}{t} = \frac{1500}{50} = 30$

TAKE $\frac{\sigma_2}{\sigma_1} = 0$ (CONS.)

→ $\lambda_p = 0.529 < 0.673$

→ $\rho = 1.0$

— BENDING : $S = \frac{bh^2}{6} = \frac{50(2400)^2}{6} = 4.8 \times 10^7$

→ $\sigma_{max} = \frac{7.5 \times 10^9}{4.8 \times 10^7} = 156$ MPa.

— SHEAR : $A_v = 50(1500) = 7.5 \times 10^4$

→ $\sigma_{v\,max} = \frac{10 \times 10^6 N}{7.5 \times 10^4} = 133$ MPa.

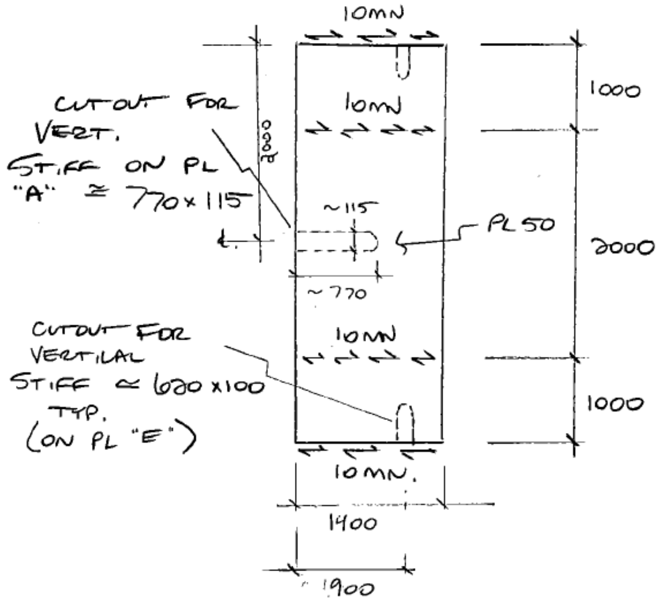
AS A ROUGH CHECK :

WORST CASE VON MISES STRESS (USE MAX + ≥ CONSERVATIVE)

→ $\sigma_{v.m.} = \left[133^2 + 133(156) + 156^2 + 3(133)^2 \right]^{\frac{1}{2}} = 340$ MPa $< \frac{460}{1.05}$

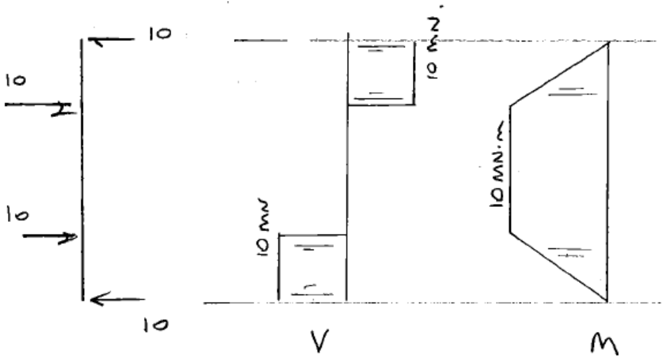
$= 438$ O.K.

→ CHECK HORIZONTAL PL 50:



[$\frac{b}{t}$ SAME AS VERT. PL 50 → NO REDUCTION FOR LOCAL PLATE SLENDERNESS...]

→ TREAT AS A SIMPLE BEAM:



→ V_{max} = 10 MN
M_{max} = 10 MN·m.

→ MAX SHEAR STRESS $\simeq \frac{10 \times 10^6 N}{(850)(50)}$ = 235 MPa

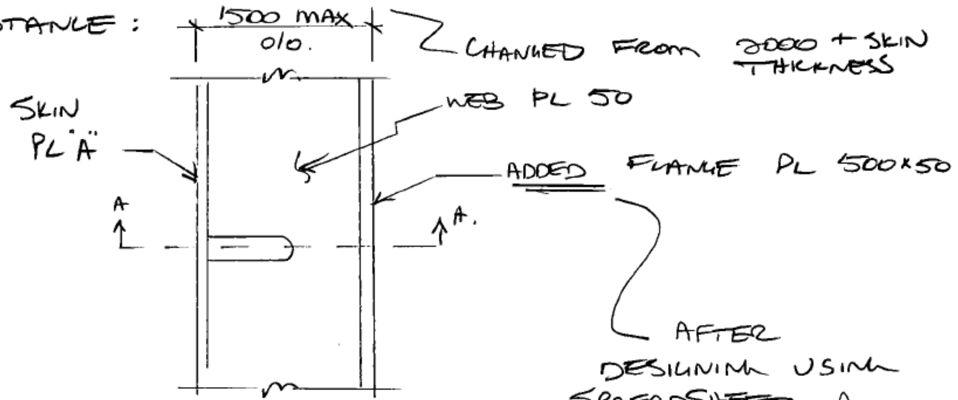
→ $\frac{460}{1.05\sqrt{3}}$ = 253 MPa ⇒ OK ✓ Σ AREA ON OUTSIDE OF CUTOUT FOR VERT. STIFF ON PL "E".

(w̄ PL 50, SHEAR BUCKLING NOT AN ISSUE ON 1350 WEB)

→ BENDING STRESS ⇒ MAX B/W LOADS FROM VERTICAL PL'S 50...

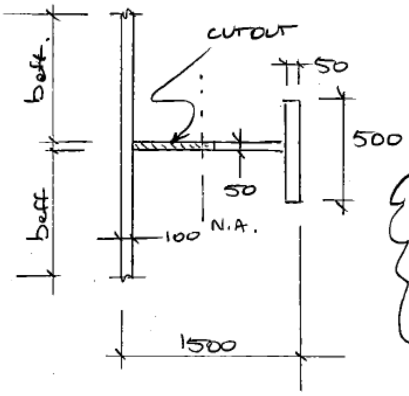
⇒ USING SPREADSHEET {FINAL SPREADSHEET CHECK ATTACHED AT THE END OF THESE CALC. NOTES}.

→ SECTION USED TO COMPUTE BENDING RESISTANCE:



AFTER DESIGNING USING SPREADSHEET, A 500 x 50 FLANGE IS REQ'D TO PROVIDE ADEQUATE RESISTANCE

→ SECTION A-A:



(2010 OCT 15)
→ DOUBLE 500x50 FLANGES REPLACED W SINGLE FULL DEPTH 3000 x 50 FLANGE — SEE DESIGN DRAWINGS.

→ TO COMPUTE b_{eff}, USE EN 1993-1-5 §3.2.1
{TABLE 3.1}

→ $A_{sl} = 0$ → $\alpha_0 = 1$ → TAKE $b_0 = 1600$

$L_e = 4000$ → $K = \frac{1600}{4000} = 0.4$

$\beta = \frac{1}{1 + 6.4(0.4)^2} = 0.494$ → $b_{eff} = 0.494(1600)$
$= 791$ mm.

→ SPREADSHEET COMPUTES I AND S FOR EXTREME FIBRE @ SKIN PL + FLANGE PL.
AT CUTOUT SECTION W N.A. COMPUTED ON BASIS OF GROSS SECTION...

→ PEAK STRESS IN SKIN (σ_{TRANS}) FROM BENDING.

$= \frac{M}{S_{SKIN}} = 57$ MPa (SPREADSHEET)

(ON SECTION W WEB CUTOUT...)

→ TO OBTAIN PEAK VERTICAL STRESS IN SKIN PL, RESULTS OBTAINED FROM MJK:

→ SICILIA TOWER: AT EL 40 m $\sigma_{LONGIT} = 418.41$ (MAX)

AT EL. 55m $\sigma_{LONGIT} = 386.45$ (MAX)

→ BUFFER CONN. IS AT 54.190m EL. (MAY CHANGE)

→ INTERPOLATING, σ_{LONGIT} @ EL 54.190m

$\approx$ 388.2 MPa.

→ $\sigma_{CRITICAL}$ = 438 MPa (MJK RESULTS)

∴ CHECK COMBINED WORST CASE STRESSES IN SKIN USING VON MISES:

→ $\sigma_{v.m.} = \left[\sigma_{LONGIT}^2 - \sigma_{LONGIT}\sigma_{TRANS} + \sigma_{TRANS}^2\right]^{\frac{1}{2}}$

$= \left[(388.2)^2 + (388.2)(-5.7) + (-5.7)^2\right]^{\frac{1}{2}} = 419$ MPa

< 438 MPa.

→ O.K. ✓

(2010 OCT. 15) FURTHER S/S → SEE LATER CALC'S

∴ CHANGES TO TENDER DESIGN:

→ PL120 RADIUS CHANGED TO 600 (FROM 500) AROUND PIN HOLES → PL WIDTH INCREASED BY 100 mm.

→ HORIZONTAL PL 50 (2) WIDTH DECREASED FROM 2000 TO 1500 TOTAL (FROM FACE OF SKIN PL)

→ SKIN IS UNCHANGED AT 100 THICK, FLANGE 500 X 50 ADDED, HORIZ PL. 50 WIDTH DECREASED TO 1350.

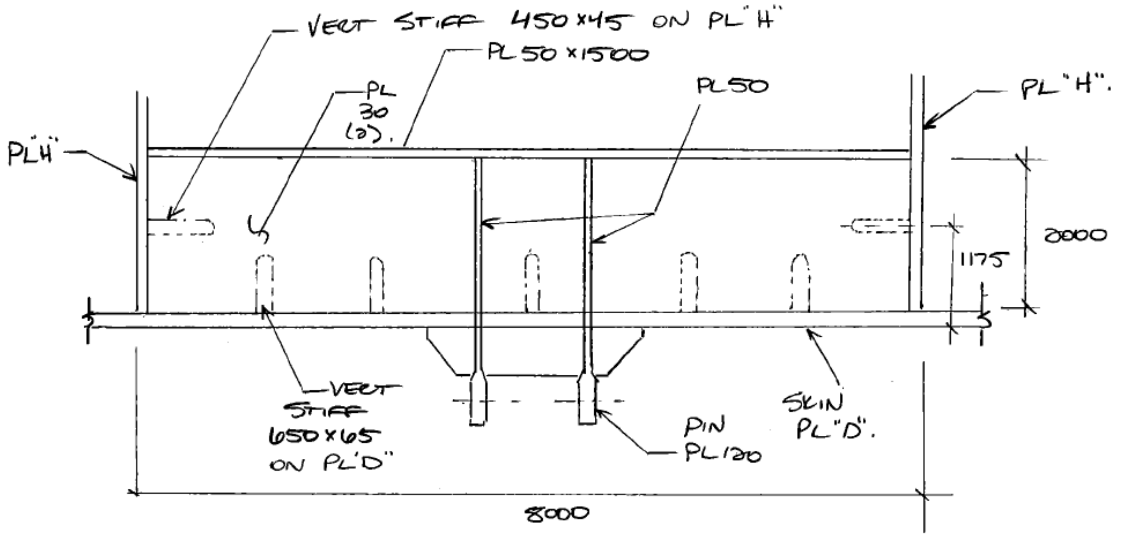
—— NOTE: CHECK WELDS B/W VERTICAL PL50 AND HORIZONTAL PL 50 → L_{WELD} = 2(1350) = 2700 mm PER CONNECTION → LOAD = 10 MN / CONNECTION

→ WITH $\frac{a15}{a15}$ ▷ → PROVIDES 3.75 kN/mm/WELD.

→ 2700(3.75) = 10125 kN > 10 MN → JUST WORKS

HOWEVER THERE IS THE WELDS TO SKIN AS WELL → SHOULD BE MORE THAN ADEQUATE.

→ TRANSVERSE BUFFER CONNECTION:

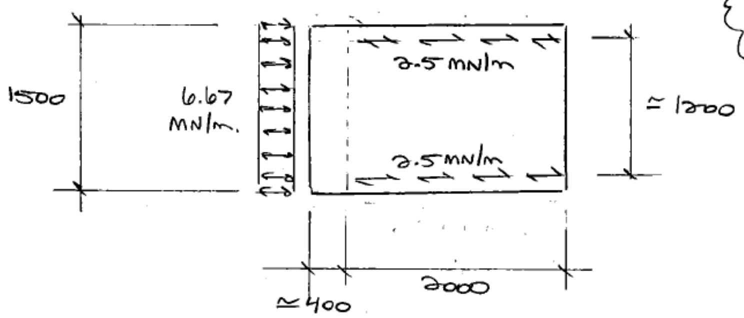


→ PIN PL 120 ⇒ SEE CALCS P 2 (R500 INCREASED TO R600).

S/S (2010 OCT 15) → PIN & CHANGED - SEE LATER CALCS + DESIGN DWGS.

→ VERTICAL PL'S 50:

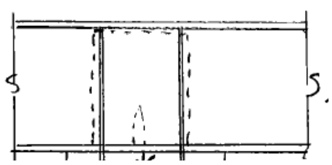
→ CONTACT STRESS:



$$\rightarrow \sigma_{CONTACT} = \frac{10\ MN}{1500(50)} = 133\ MPa.$$

→ SHEAR STRESS ON PL 50 ⇒ $\frac{5\ MN}{(1500)(50)} = 67\ MPa.$

→ CHECK TEAROUT SHEAR STRESS ON PL 30 "WEBS":



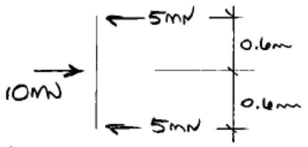
→ TOTAL FORCE = 20 MN ON 4 – 30 WEB SECTIONS.

→ $\sigma_v = \dfrac{20\times10^6}{4(30)(2000)} = 83$ MPa

→ BENDING ON VERTICAL PL 50 (NOT LIKELY FOR LOAD TRANSFER, BUT CHECK OUT OF CURIOSITY...)

{b/t IS O.K. — $\rho = 1.0$}.

→ ASSUME:



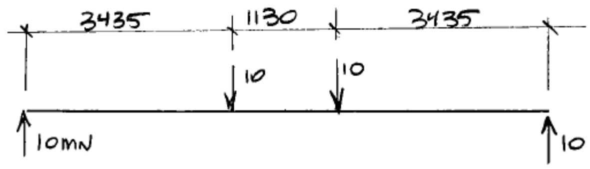
⇒ $M_{max} = \dfrac{PL}{4} = 3$ MN·m.

→ $S \simeq \dfrac{(2400)^2(50)}{6} = 4.8\times10^7$ ⇒ $\sigma_{max} = 63$ MPa

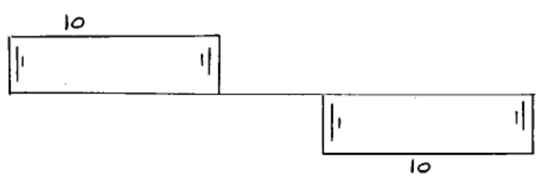
→ CHECK HORIZONTAL "WEB" PL's 30:

→ SPREADSHEET USED, SAME PROCEDURE AS FOR LONGIT. BUFFER CONNECTION:

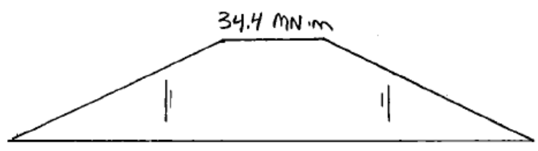
→ DEMANDS COMPUTED ASSUMING:



V:



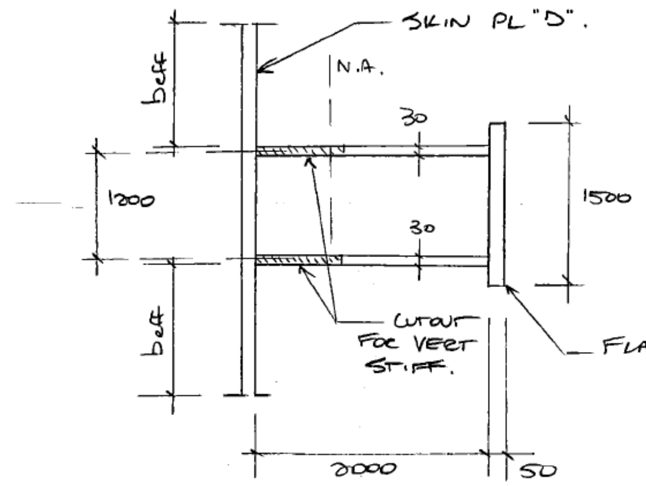
M:



⇒ RESISTING CROSS-SECTION IS AS SHOWN ON THE FOLLOWING PAGE:

{CALCULATION OF b_{eff} SIM. TO CALC'S p5 FOR LONGIT. BUFFER}

→ SECTION



⇒ FLANGE b/t's
→ OUTSTAND — NO PROBLEM
→ INTERNAL → $\frac{\sigma_2}{\sigma_1} = 1$
$k_\sigma = 4$ ⇒ $\lambda_p = 0.562$
< 0.673
→ $\rho = 1.0$
O.K. ✓

→ PEAK TRANS. SKIN STRESS (FROM SPREADSHEET @ CUTOUT SECTION) → $\sigma_{TRANS\ (MAX)} = 79$ MPa (SICILIA TOWER)

→ LONGIT. STRESS (VERTICAL) BASED ON INTERPOLATING MSK RESULTS (SEE CALCS P6) → $\sigma_{LONGIT\ (MAX)} = 346$ MPa (SICILIA).

→ VON MISES COMBINED STRESS
$$= \left[(79)^2 + (79)(346) + (346)^2\right]^{\frac{1}{2}} = 392 \text{ MPa}$$

$\sigma_{CRITICAL} = 427$ FOR THIS SKIN PL.
→ $392 < 427$ ⇒ O.K. ✓.

⇒ FOR CALABRIA TOWER, SKIN PL IS ONLY 40 THICK SINCE BUFFERS @ HIGHER ELEV. (IN SEGMENT 4)
{NOTE THAT EL. OF BUFFER CONN'S MAY CHANGE}

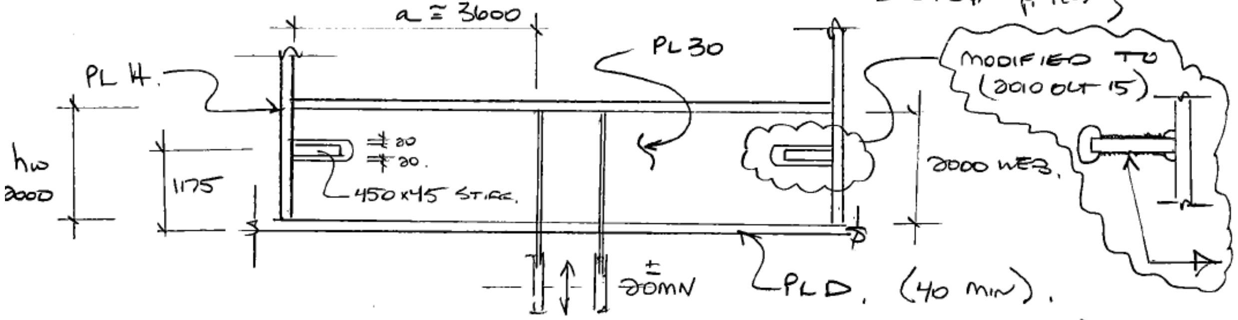
→ FOR CALABRIA, TO MAKE THIS ARRANGEMENT WORK, MUST INCREASE PL "D" SKIN PL THICKNESS TO 45mm (FROM 40).

∴ CHANGES TO TENDER DESIGN: (2010 OCT 15) SK...
→ PL 120 (PIN PL) — RADIUS INCREASED TO 600 (FROM 500)
→ ALSO — MSK TO CHANGE SEGMENT 4 PL "D" IN CALABRIA FROM 40 TO 45

SHEAR @ PLH: ≈ 1150 CONN. LENGTH (LONG.) ⇒ $\sigma_v = 10 \times 10^6 / [2(1150)(30)] = 145$ MPa — O.K. ✓

→ DOUBLE - CHECK SHEAR CAPACITY ON WEBS OF TRANSVERSE BUFFER AT CUTOUT FOR VERTICAL STIFFENERS:

{ALSO SEE ATTACHED SKETCH p. 9(c)}



→ ASSUME ALL SHEAR MUST BE TAKEN BY REMAINING WIDTH OF PL 30 TO THE INSIDE OF 450 x 45 STIFF ON PL H: (CONSERVATIVE).

→ LENGTH OF CONTACT W PL H

$$\leq 2000 + 40 - 1175 - \frac{45}{2} - 20$$

$$= 822 \text{ mm.}$$

→ SHEAR STRESS FROM 10 MN SHEAR DEMAND

$$= \frac{10 \times 10^6 \text{ N}}{2(30)(822)} = 203 \text{ MPa.}$$

↖ 2 WEB PL's 30 ...

→ CHECK SHEAR BUCKLING REDUCTION OF WEB PL:

$$\frac{a}{h_w} = \frac{3600}{2000} = 1.8 > 1.0 \Rightarrow k_\tau = 5.34 + 4\left(\frac{2}{3.6}\right)^2 = 6.57$$

$$\rightarrow \bar{\lambda}_w = \frac{2000}{37.4(30)(0.715)\sqrt{6.57}} = 0.972 > 0.83 \text{ AND } < 1.08$$

$$\rightarrow \chi_w = \frac{0.83}{0.972} = 0.85$$

{TABLE 5.1 — EN 1993-1-5 §5.3}.

$$\therefore \text{MAX ALLOWABLE SHEAR STRESS} \cong \frac{460(0.85)}{1.05\sqrt{3}} = 216 \text{ MPa.}$$

→ 203 < 216 ⇒ SHOULD BE OK ...

→ NOTE: IF WE LOOK AT ENTIRE SECTION AT CUTOUT, WE HAVE
2000 WIDTH − 45 − 2(20) = 1915 FOR SHEAR

→ AVG SHEAR STRESS = $\frac{10 \times 10^6}{2(30)(1915)}$
= 87 MPa —— NO PROBLEM. ✓

→ CHECK DEMAND ON WELDS:

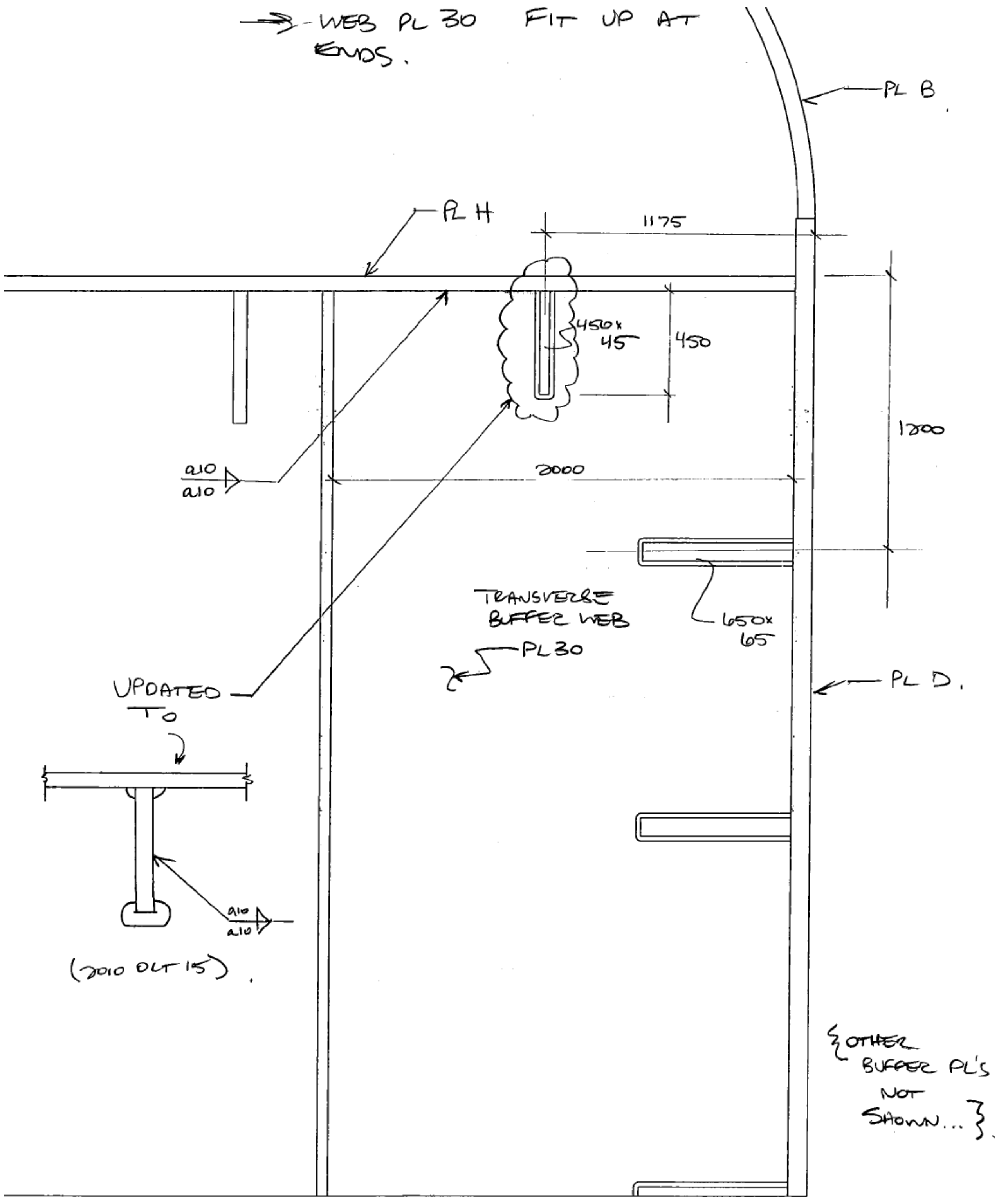
→ IF WE USE ONLY PORTION INSIDE OF VERT. STIFFENER,
⇒ $L_{WELD} \cong 4(822) = 3288$ mm.
⇒ DEMAND = $\frac{10 \times 10^3 \text{ kN}}{3288 \text{ mm}}$ = 3.04 kN/mm.

⇒ a13 ▽ PROVIDES 3.25 kN/mm. — BARELY ADEQUATE.

→ TAKING TOTAL LENGTH OF WELDS HERE:
⇒ = 2(2)(1915) ⇒ L_{WELD} = 7660
⇒ $\frac{10\,000}{7660}$ = 1.31 kN/mm.
⇒ a6 ▽ PROVIDES 1.50 kN/mm.

→ USING a10/a10 AS PREVIOUS PROVIDES
2.50 kN/mm > 1.31 ⇒ GOOD − USE THIS.

NOTE: WEB PL 30 IS NOW WELDED TO LONGIT. STIFF. ON PLATE H @ EITHER END TO ENSURE SHEAR TRANSFER.
(2010 OCT 15) ⇒ SEE NEXT PAGE SKETCH.



WEB PL 30 FIT UP AT ENDS.
PL B.
PL H
1175
450x 45
450
1200
a10 a10
2000
TRANSVERSE BUFFER WEB
PL 30
650x 65
PL D.
UPDATED TO
a10 a10
(2010 OCT 15).
{OTHER BUFFER PL'S NOT SHOWN...}.

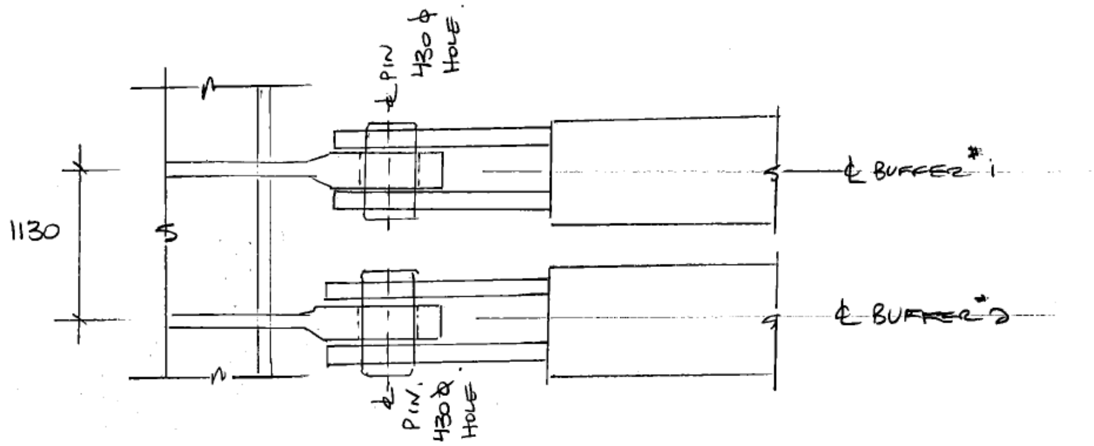
INVESTIGATE EFFECTS OF CHANGES MADE TO TRANSVERSE BUFFER CONNECTION:

→ BUFFERS (2) ARE NOW CONNECTED TO THE TOWER LEG BY A SINGLE PIN.

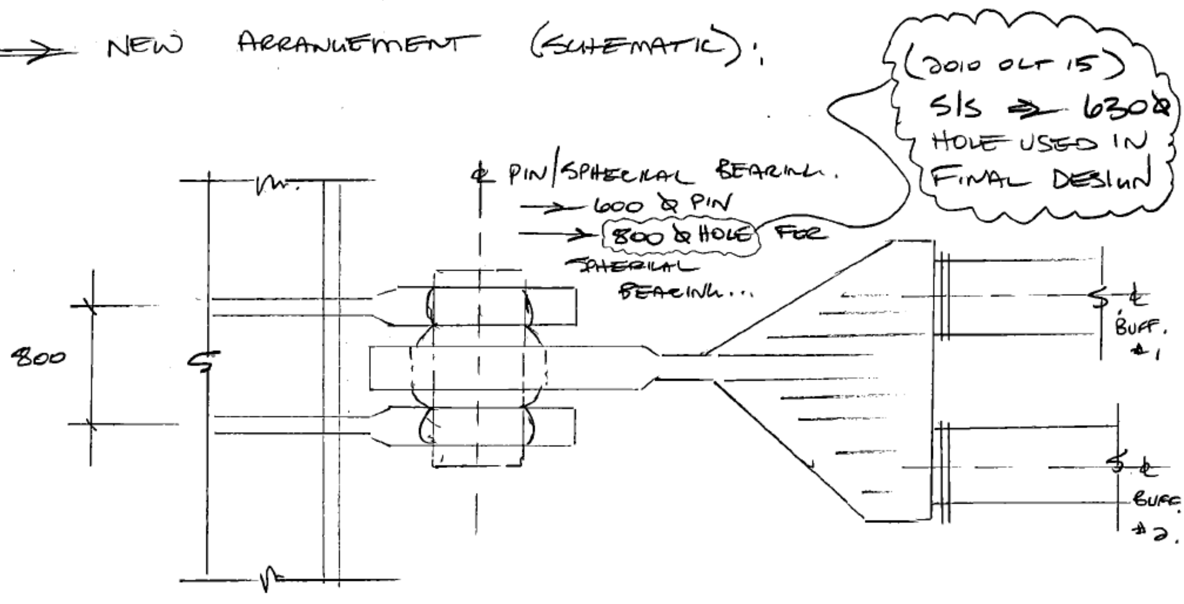
→ ASSUME TOTAL LOAD IS THE SAME, AT 10MN / BUFFER = 20MN TOTAL.

→ ALL OF THIS LOAD IS NOW DELIVERED TO THE TOWER BY ONE PIN, HOWEVER, THERE ARE STILL 2 PIN PLATES.

→ PREVIOUS ARRANGEMENT (SCHEMATIC):



→ NEW ARRANGEMENT (SCHEMATIC):



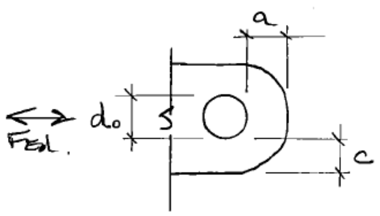
→ CHECK REQUIRED END + EDGE DISTANCES ON PIN PLATE, AS WELL AS MINIMUM t:

→ EN 1993-1-8 : 2005

§ 3.13.1 — TABLE 3.9 :

FOR $t = 120$ mm :

$F_{Ed} = 10$ MN | PLATE 120



→ $a \geq \frac{F_{Ed}\gamma_{M0}}{2tf_y} + \frac{2d_0}{3}$

$= \frac{10 \times 10^6 (1.05)}{2(120)(460)} + \frac{2(800)}{3} = 629$ mm MIN

→ $c \geq$ " $+ \frac{(800)}{3} = 362$

(SUPERSEDED (S/S) → AN Ø 630 USED IN FINAL... (2010 OCT 15))

→ CHECK BEARING RESISTANCE OF THE PIN :

→ TABLE 3.10 :

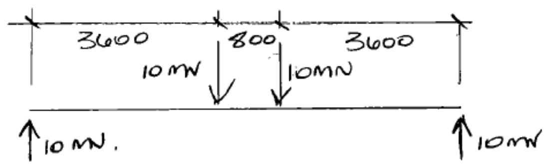
$F_{b,Rd} = \frac{1.5tdf_y}{\gamma_{M0}} = \frac{1.5(120)(800)(460)}{1.05(10^6)} = 63$ MN

> 10 MN

→ O.K. ✓.

→ COMPUTE PEAK BENDING IN WEB PL 30'S :

→ SAME ASSUMPTIONS AS PREVIOUSLY, BUT FORCES ARE CLOSER TOGETHER...



→ $M_{MAX} = 36$ MN·m

→ CHECK USING SPREAD SHEET...

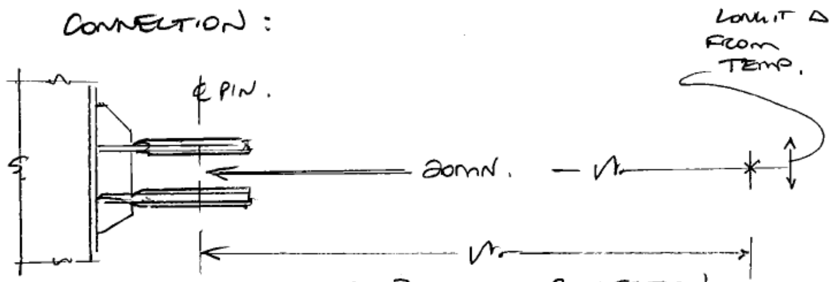
→ SKIN PL D (CALABRIA) SHOULD INCREASE THICKNESS TO 50 mm FOR VON MISES — SEE ATTACHED SPREADSHEET.

(SICILIA STILL OK W/ 55 PL.)

→ NOTE: MUST RECHECK WITH NEW PLATE DEMANDS IN TOWER WHEN

→ MAY NEED TO CHECK OUT OF PLANE STABILITY OF CONNECTION:

→



→ ESTIMATE OF DECK EXPANSION | CONT. FROM ΔTEMP.

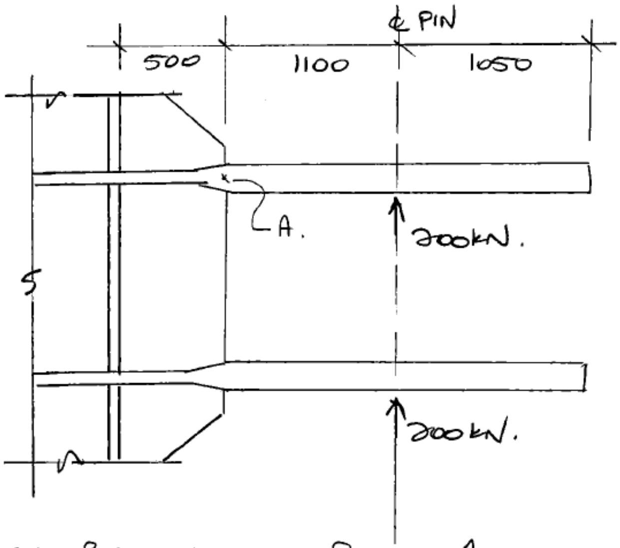
→ SAY 30°C OVER 1/2 MAIN SPAN:

→ $\Delta_{LONGIT} \simeq (1.2\times10^{-5}/°C)(30°C)(1650\times10^{3}\ mm)$

$\cong 600\ mm.$

→ $\frac{600}{30000} = 2\%.$ ∴ TRY A "KICK" OF 2% OF BUFFER FORCE = 400 kN.

→ CONSERVATIVELY, APPLY KICK AS FOLLOWS:



→ CHECK BENDING @ POINT A:

→ $M_f = 200(1.1) = 220\ kN{\cdot}m.$

→ M_r → PL 120 × 1600 → $S = \frac{120^2(1600)}{6}$

$= 3.84\times10^{6}\ mm^3.$

→ $\sigma_{max} = \dfrac{220 \times 10^6}{3.84 \times 10^6} = 57.3$ MPa.

AXIAL STRESS HERE $\cong \dfrac{10 \times 10^6 \text{ N}}{120\,(1600)} = 52.1$ MPa.

→ $\sigma_b + \sigma_{axial} \cong 110$ MPa → NO PROBLEM.

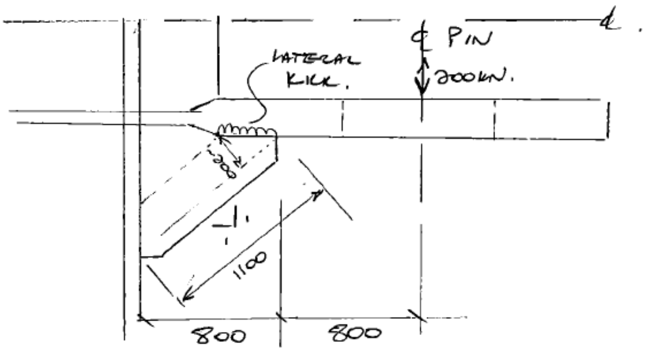
→ HOWEVER, THIS ASSUMES A RIGID SUPPORT FOR THE PIN PL 120 AT THIS POINT
→ NOT LIKELY . . .

∴ EXTEND THE PL 30 HORIZONTAL CORNER STIFFS TO PICK UP THE LATERAL KICK BY THE END OF THE PL 120 (WHERE 120 STARTS TO TAPER INTO 50 PL)

→ TRY THE FOLLOWING:

→ EXTEND PL 30 CORNERS BY 300 E.S. AND INCREASE TO PL 40 FOR STIFFNESS:

→



→ CHECK CAPACITY OF PLATE TO CARRY KICK TO SKIN → TREAT AS COLUMN 40 x 200 x 1100 Lk.
(VERY CONSERVATIVE)

→ FORCE INCREASES ALONG COLUMN AXIS TO $\simeq \sqrt{2}\,(200 \text{ kN}) = 282$ kN.

→ $\sigma_{cr,c} = \dfrac{\pi^2\,(200\,000)(40)^2}{12\,(1100)^2} = 218$ MPa.

$$\longrightarrow \bar{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cr,c}}} = \sqrt{\frac{460}{218}} = 1.45$$

$$\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] \qquad \{\alpha = 0.21\} \text{ EN1993-1-5 § 4.5.3.}$$

$$= 0.5\left[1 + 0.21(1.45 - 0.2) + (1.45)^2\right] = 1.69$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{1.69 + \sqrt{(1.69)^2 - (1.45)^2}} = 0.392$$

∴ COMPRESSIVE CAPACITY REDUCED FOR BUCKLING..

$$\doteq \frac{0.392(460)(40)(200)}{1.05(10^3)} = 1375 \text{ kN} > 282 \text{ kN}$$

→ COULD LIKELY SLIM THIS DOWN, HOWEVER USE 40 PL TO ENSURE ADEQUATE STIFFNESS TO ATTRACT THE OUT OF PLANE LOAD... (VERY SMALL QUANTITY RELATIVE TO ITS IMPORTANCE).

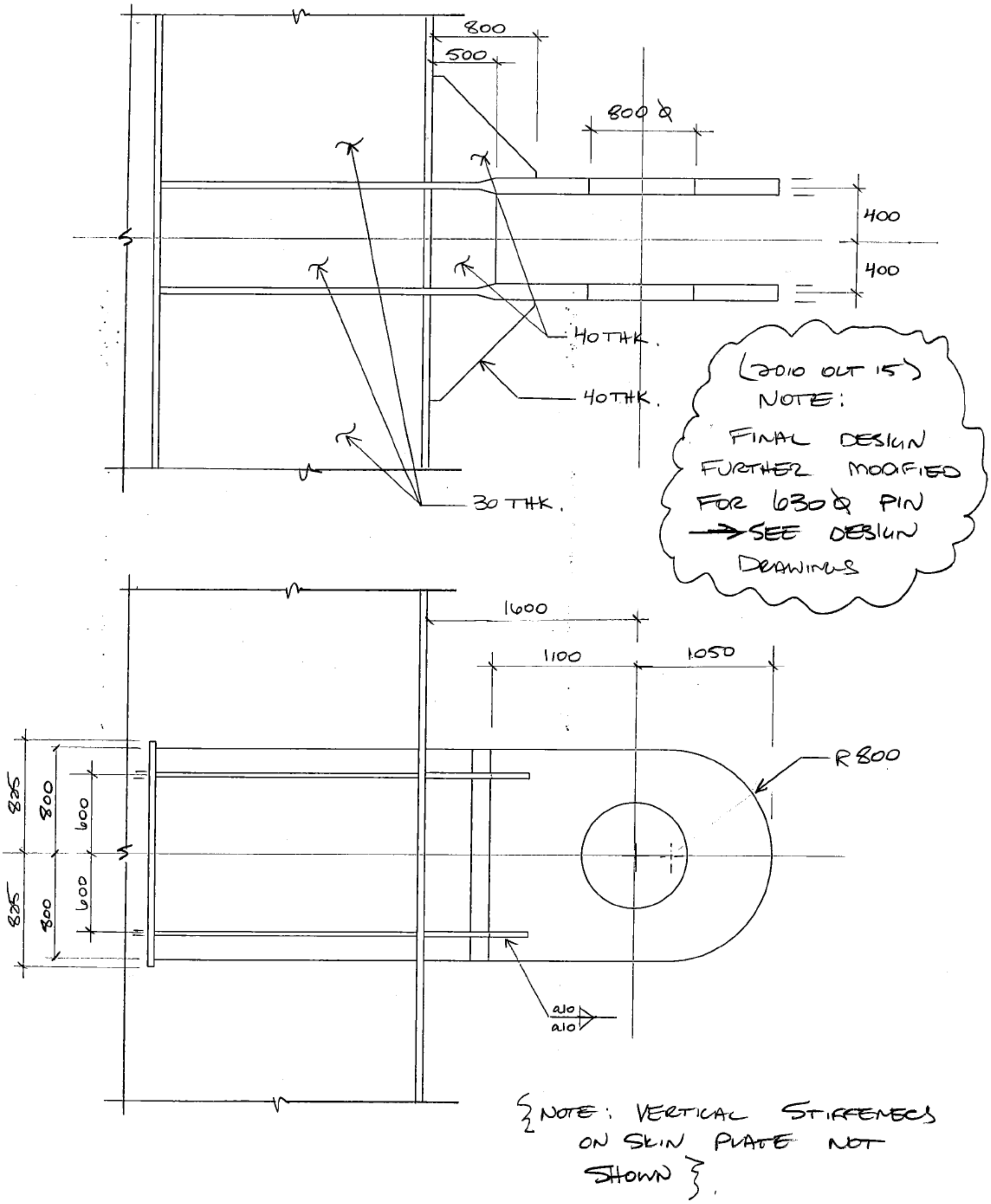
→ SEE ATTACHED SKETCHES FOR FINAL LAYOUT.

→ CHECK WELDS B/W PL 40 EXTENSION AND PL 120:

→ REQUIRED CAPACITY : $L_{WELD} \simeq 500$ mm/CONN.

$$\rightarrow \frac{287 \text{ kN}}{500} \leq 0.5 \text{ kN/mm} \rightarrow \text{EASILY TAKEN BY } \frac{8}{8}\triangleright$$

→ MODIFICATIONS TO PREVIOUS DESIGN...



———— MODIFY PIN PLATES FURTHER ON TRANSVERSE BUFFER BASED ON NEW INFO FROM CRS

→ PIN HOLE DIA. IN PIN PLATES NOW REDUCED FROM 800 Ø TO 630 Ø

→ RESIZE PL 120 USING EN1993-1-8 TABLE 3.9:

$$a \geq \frac{10 \times 10^6 (1.05)}{2(120)(460)} + \frac{2}{3}(630) = 515 \text{ mm.} \quad (\text{USE } 520)$$

$$c \geq \quad " \quad + \frac{1}{3}(630) = 305 \text{ mm.}$$

→ ALSO: ADJUST LENGTH OF PROJECTING PIN PLATES TO BE CLOSER TO SKIN PLATE

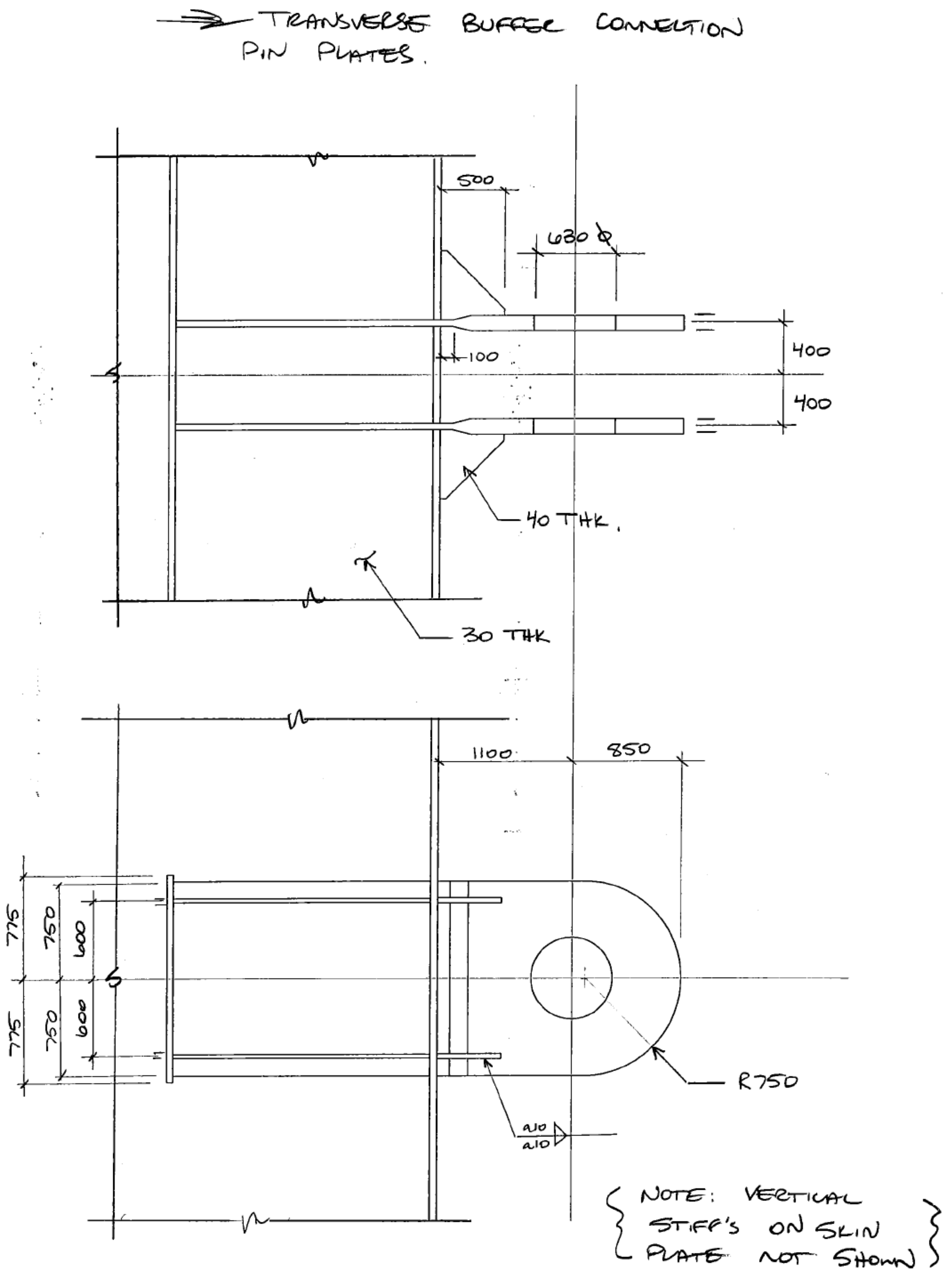
→ MIN DISTANCE IS 1100 {1000 FOR CONNECTING PIN PL + 100 GAP}.

→ SKETCH UP IN CAD ← SEE ATTACHED.

→ CHECK OUT OF PLANE FORCE @ 2% OF BUFFER LOAD = 400 kN TOTAL ⇒ 200 kN / PLATE. @ ℄ PIN:

→ DUE TO TIME CONSTRAINT, USE STIFFENING PL40 ES. SIM TO PREVIOUS DESIGN

→ THIS IS CONSERVATIVE SINCE DEMANDS ON PL40 WILL BE SMALLER (LESS ECCENTRICITY OF LATERAL LOAD).



TRANSVERSE BUFFER CONNECTION
PIN PLATES.
500
630 Φ
100
400
400
40 THK.
30 THK
1100
850
775
750
600
775
750
600
R750
a10
a10
NOTE: VERTICAL STIFF'S ON SKIN PLATE NOT SHOWN

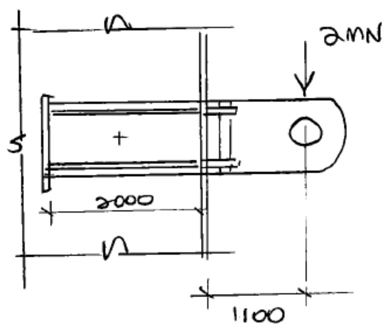
→ ALSO, CHECK RESISTANCE TO VERTICAL COMPONENT OF BUFFER LOADS:

→ FROM CRS, REACTION FROM WEIGHT OF TRANSVERSE BUFFER @ ℄ PIN ≅ 1 MN.

→ ASSUME VERTICAL DECK DEFLECTION OF 1.5m → $\frac{1.5m}{30m \text{ LENGTH}}$ (≈) = 5%

→ 5% OF 20MN = 1MN.

∴ CONSERVATIVELY TAKE TOTAL VERTICAL RXN @ ℄ PIN = 1+1 = 2MN.



→ M_{max} @ FACE OF SKIN = 2200 kN.m

→ RESISTED BY 2 PLATES 50x 1500

→ $S = \frac{2(50)(1500)^2}{6} = 3.75 \times 10^7$

→ MAX BENDING STRESS = $\frac{2200 \times 10^6}{3.75 \times 10^7}$ = 58.7 MPa

→ AXIAL STRESS = $\frac{20 \text{ MN}}{2(50)(1500)}$ = 133 MPa

→ σ_{TOTAL} = 192 MPa (AXIAL)

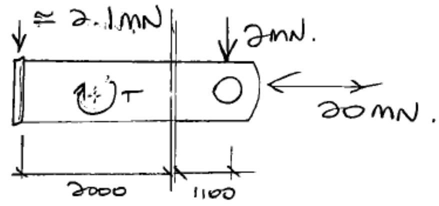
→ SHEAR FROM 2 MN = $\frac{2 \times 10^6}{50(1500)(2)}$ = 13 MPa NEGLIGIBLE.

∴ PL 50 O.K. ✓

→ CHECK TORSION ON BOX SECTION: (TRANSVERSE)

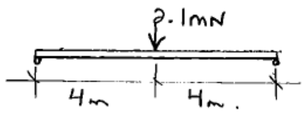
→ AS A ROUGH CHECK, SEE IF FLANGE CAN BALANCE 2 MN RXN VIA BENDING:

→



T ~~FROM~~ FROM 2MN ↓
≃ 2(2.1) = 4.2MN.m.

→ ON 8m SPAN, FLANGE BENDING APPROXIMATED AS



$M_{max} = \frac{PL}{4} = 4200$ kN.m.

$V_{max} = 1050$ kN.

FLANGE IS 50 x 1550

→ $\sigma_{BENDING} = \dfrac{4200 \times 10^6}{\frac{50(1550)^2}{6}} = 210$ MPa.
(FROM 2MN LOAD)

$\sigma_{SHEAR} = \dfrac{500 \times 10^3}{50(1550)} = 13$ MPa (NEGLIGIBLE)
(FROM 2 MN LOAD)

FLANGE STRESS FROM BENDING DUE TO 20MN LOAD (FROM SPREADSHEET)
= 175 MPa.

∴ $\sigma_{VON\ MISES} = \left[210^2 + (210)(175) + 175^2\right]^{\frac{1}{2}} = 334$ MPa
→ O.K {<< 438 MPa}.

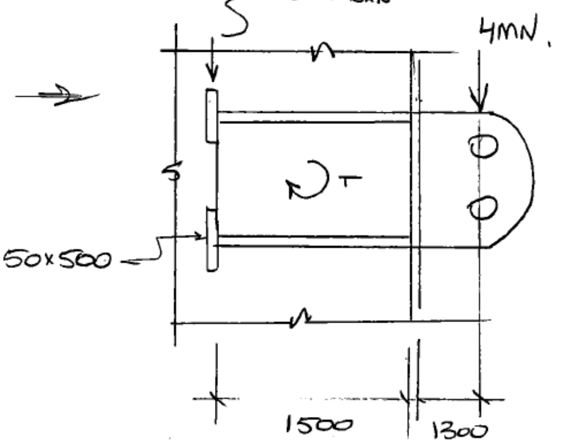
→ USE SAME PROCEDURE TO CHECK LONGITUDINAL BUFFER FOR VERTICAL LOAD:

→ ASSUME DOUBLE THE LOAD, SINCE 4 BUFFERS INSTEAD OF 2 ⇒ 4MN ↓

BY INSPECTION, PL 50 VERTICAL O.K. (3000 DEEP VS 1500)

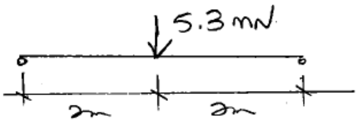
→ CHECK FLANGES :

→ FRXN FLANGES.



T ↻ FROM 4MN ↓
$\simeq 4(2m) = 8 MN \cdot m$.

→ FRXN FLANGES $\cong$ 8 MN·m / 1.5m $\cong$ 5300 kN



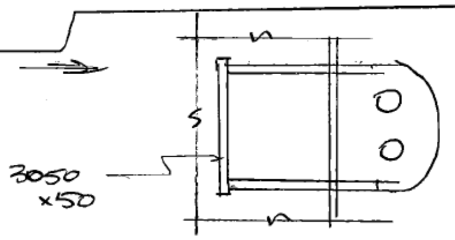
→ $M_{max} = \frac{5300(4)}{4(2)}$
= 2650 kN·m / FLANGE.

→ V_{MAX} = 1325 kN / FLANGE

FLANGES ARE 50 x 500 EA.:

→ $\sigma_{BEND} = \frac{2650 \times 10^6}{\left(\frac{50(500)^2}{6}\right)} = 1272$ MPa → NO GOOD.

→ TRY A SOLID FLANGE ACROSS THE BACK 3050 x 50



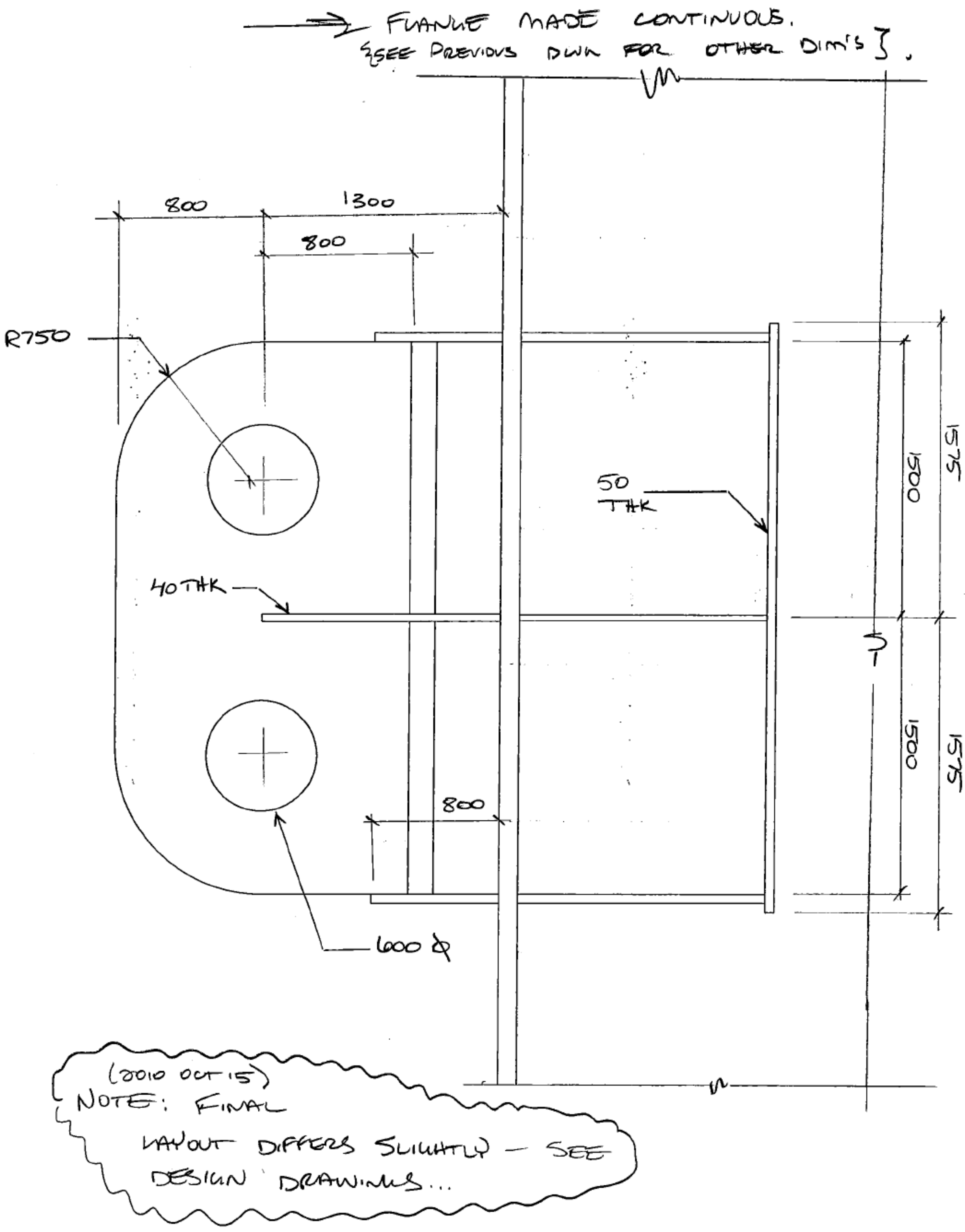
MODIFY DRAWINGS FOR THIS...

→ $\sigma_{BEND} = \frac{5300 \times 10^6}{\left[\frac{50(3050)^2}{6}\right]} = 68$ MPa

$\sigma_{SHEAR} = \frac{2650 \times 10^3}{50(3050)} = 17$ MPa — NO PROBLEM.

σ_{max} FROM BENDING BY 40MN BUFFER LOAD = 169 MPa (SPREADSHEET)

$\sigma_{VON\ MISES} = \left[68^2 + 169(68) + 169^2\right]^{1/2} = 211$ MPa → O.K. ✓



FLANGE MADE CONTINUOUS.
{SEE PREVIOUS DWG FOR OTHER DIM'S}.
800
1300
800
R750
50 THK
1500
1575
40 THK
1500
1575
800
600 ⌀
(2010 OCT 15)
NOTE: FINAL LAYOUT DIFFERS SLIGHTLY – SEE DESIGN DRAWINGS...

FURTHER UPDATE TO LONGIT. BUFFER:

→ 710 ⌀ HOLE FOR PIN + SPHERICAL BEARING, MUST BE 205 mm THICK AT CONNECTION.

→ CHECK REQ'D PLATE SIZES:
EN1993-1-8 TABLE 3.9:

→ FOR 710 ⌀ W/ 105 PL,

$$a \geq \frac{10 \times 10^6 (1.05)}{2(105)(460)} + \frac{2}{3}(710) = 565$$

$$c \geq \quad " \quad + \frac{1}{3}(710) = 328$$

→ W/ 205 PL, a REDUCES TO 529 ⇒ NOT SIGNIFICANT

∴ USE PL 105 AS PIN PLATE, WITH A 40 THK FILL PLATE EACH SIDE TO FILL IT OUT TO 205 THK.

→ SEE ATTACHED SKETCH ...

→ CHECK WELD B/W CHEEK PL 40 AND PIN PL 105:

→ ASSUME FORCES DISTRIBUTE IN PROPORTION TO PLATE THICKNESS:

→ 10 MN / PIN → EACH CHEEK PLATE TAKES $\frac{40}{205}$ (10 MN) = 1950 kN

→ PERIMETER LENGTH = 4240 mm

→ WELD DEMAND = $\frac{1950}{4240}$ = 0.46 kN/mm.

→ SMALL.

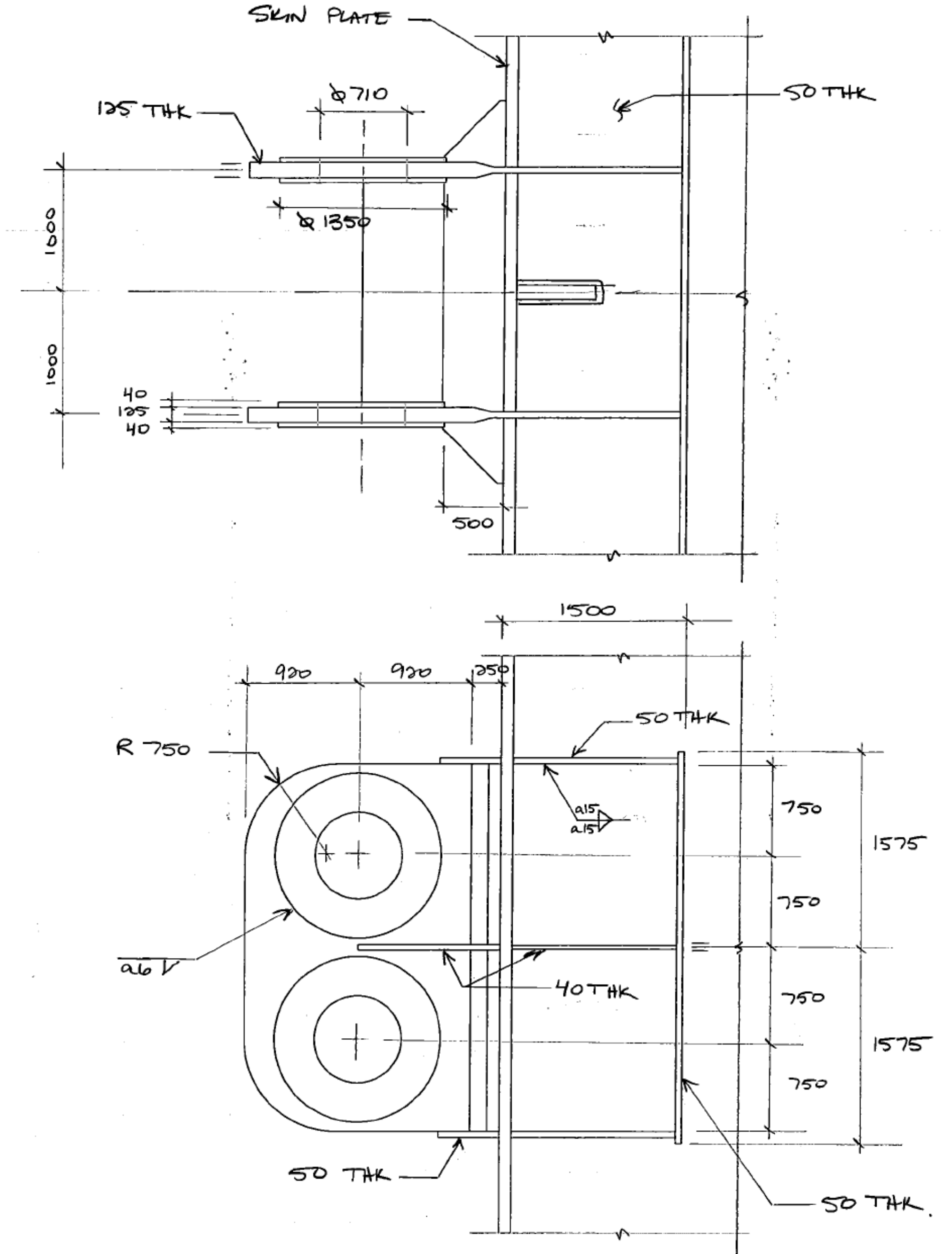
→ USE 8 ∨ AS MIN. BASED ON PLATE THICKNESS

→ a6 ∨ IS MORE THAN ADEQUATE.

→ ≃ 1.4 kN/mm ≫ 0.46

— O.K. ✓

→ FURTHER MODIFICATIONS.



FINAL DESIGN IS SIMILAR TO THIS, BUT W 600 Ø PIN HOLE → SEE DESIGN DRAWINGS.

— CHECK VERTICAL STRESSES IN SKIN PL'S @ BUFFER LOCATIONS:

→ LONGIT. BUFFERS:

- SICILIA @ EL. 55.557 → RIGHT B/W 3 + 4
- CALABRIA @ EL. 63.150 → IN SEGMENT 4

→ TRANSVERSE BUFFERS:

- SICILIA @ EL.
- CALABRIA @ EL.

} CURRENTLY NOT KNOWN EXACTLY
→ LOWER THAN LONGIT
→ BOTH IN SEGMENT 3?

⇒ PLATE SIZES:

SICILIA = 3 — A ⇒ 100, D ⇒ 50
= 4 — A ⇒ 100, D ⇒ 40

CALABRIA = 3 — A ⇒ 100, D ⇒ 55
= 4 — A ⇒ 100, D ⇒ 45

→ CHECK ALLOWABLE STRESS CAPACITIES (WITH REDUCTION FOR COLUMN + LOCAL PLATE BUCKLING) → FROM JSL SPREADSHEET...

→ SICILIA < 3A ⇒ 433 MPa, 3D ⇒ 412 MPa.
4A ⇒ 429 MPa, 4D ⇒ 378 MPa.

→ CALABRIA < 3A → 433 MPa, 3D ⇒ 424 MPa
4A → 429 MPa, 4D ⇒ 391 MPa.

NOTE: THESE HAVE BEEN MODIFIED → SEE BELOW...

⇒ APPLIED VERTICAL STRESS FROM GOVERNING LOADCASE:

→ SEE ATTACHED PRINTOUT OF STRESSES AND ALLOWABLE STRESS CAPACITIES FROM TOWER DESIGN SPREADSHEET.

→ FINAL CHECK OF SKIN STRESSES USING VON MISES IS DONE IN SPREADSHEET (ATTACHED).

SICILIA TOWER - SEGMENT 3 - LEG 1 STRESSES

6903 - Longitudinal

Plate D — Elev 40.00

Location	Stress	critical stress	UR
SP3	-363.61	438.10	0.83
SP4	-284.69	438.10	0.65
SP8	-324.36	438.10	0.74
SP9	-275.65	438.10	0.63
LS7	-379.71	434.98	0.87
LS8	-372.73	434.98	0.86
LS9	-366.89	435.31	0.84
LS10	-340.57	432.10	0.79
LS11	-325.16	421.11	0.77
LS7	-361.98	438.10	0.83
LS8	-334.75	400.60	0.84
LS9	-261.23	400.60	0.65
LS10	-297.54	438.10	0.68
LS11	-333.86	438.10	0.76

Plate A — Elev 40.00

Location	Stress	critical stress	UR
SP1	-425.03	438.00	0.97
SP2	-430.34	438.10	0.98
SP7	-419.73	438.10	0.96
SP5	-387.40	438.10	0.88
SP6	-385.68	438.00	0.88
SP10	-384.75	438.10	0.88

Plate D — Elev 55.00

Location	Stress	critical stress	UR
SP3	-336.65	438.10	0.77
SP4	-268.05	438.10	0.61
SP8	-308.55	438.10	0.70
SP9	-251.58	438.10	0.57
LS7	-357.36	434.98	0.82
LS8	-347.51	434.98	0.80
LS9	-340.40	435.31	0.78
LS10	-316.77	432.10	0.73
LS11	-303.87	421.11	0.72
LS7	-341.52	438.10	0.78
LS8	-319.90	400.60	0.80
LS9	-241.48	400.60	0.60
LS10	-273.84	438.10	0.63
LS11	-306.21	438.10	0.70

Plate A — Elev 55.00

Location	Stress	critical stress	UR
SP1	-394.63	438.00	0.90
SP2	-398.87	438.10	0.91
SP7	-390.39	438.10	0.89
SP5	-353.58	438.10	0.81
SP6	-353.47	438.00	0.81
SP10	-353.36	438.10	0.81

CALABRIA TOWER - SEGMENT 4 - LEG 1 STRESSES

6901 - mw3

Plate D — Elev 55.00

Location	Stress	critical stress	UR
SP3	-389.20	438.10	0.89
SP4	-375.25	438.10	0.86
SP8	-263.24	438.10	0.60
SP9	-261.54	438.10	0.60
LS7	-372.67	426.02	0.87
LS8	-376.86	426.02	0.88
LS9	-385.62	426.42	0.90
LS10	-385.09	425.27	0.91
LS11	-382.10	405.77	0.94
LS7	-341.25	421.78	0.81
LS8	-332.22	362.95	0.92
LS9	-318.75	362.95	0.88
LS10	-327.16	421.78	0.78
LS11	-336.99	437.78	0.77

6903 - Longitudinal

Plate A — Elev 55.00

Location	Stress	critical stress	UR
SP1	-411.07	438.00	0.94
SP2	-417.20	438.10	0.95
SP7	-405.56	438.10	0.93
SP5	-420.39	438.10	0.96
SP6	-425.62	438.00	0.97
SP10	-431.40	438.10	0.98

Plate D — Elev 71.00

Location	Stress	critical stress	UR
SP3	-344.57	438.10	0.79
SP4	-330.20	438.10	0.75
SP8	-267.63	438.10	0.61
SP9	-270.83	438.10	0.62
LS7	-343.95	426.02	0.81
LS8	-341.77	426.02	0.80
LS9	-343.07	426.42	0.80
LS10	-340.20	425.27	0.80
LS11	-337.17	405.77	0.83
LS7	-325.18	421.78	0.77
LS8	-317.06	362.95	0.87
LS9	-303.08	362.95	0.84
LS10	-311.82	421.78	0.74
LS11	-320.57	437.78	0.73

Plate A — Elev 71.00

Location	Stress	critical stress	UR
SP1	-377.63	438.00	0.86
SP2	-380.82	438.10	0.87
SP7	-374.97	438.10	0.86
SP5	-390.64	438.10	0.89
SP6	-392.68	438.00	0.90
SP10	-395.18	438.10	0.90

Buffer Connection Check

		Sicilia - Longit Buffer	Sicilia - Transverse Buffer	Calabria - Longit Buffer	Calabria - Transverse Buffer
		Plate A - 4 meter span USING SKIN	Plate D - 8 meter span USING SKIN	Plate A - 4 meter span USING SKIN	Plate D - 8 meter span USING SKIN
Skin t	(mm)	100	50	100	45
epsilon		0.715	0.715	0.715	0.715
15 epsilon t		1072	536	1072	482
a0		1	1	1	1
b0	(mm)	1600	2000	1600	3000
Le	(mm)	4000	8000	4000	8000
K		0.40	0.25	0.40	0.38
beta		0.49	0.71	0.49	0.53
b eff	(mm)	791	1429	791	1579
Skin w	(mm)	1631	4087	1631	4388
Web w	(mm)	1350	2000	1350	2000
Web t	(mm)	50	60	50	60
Flange w	(mm)	1575	1500	1575	1500
Flange t	(mm)	50	50	50	50
Total Depth	(mm)	1500	2100	1500	2095
A Skin	(mm^2)	163103	204357	163103	197455
d Skin	(mm)	50	25	50	22.5
A Web	(mm^2)	67500	120000	67500	120000
d Web	(mm)	775	1050	775	1045
A Flange	(mm^2)	78750	75000	78750	75000
d Flange	(mm)	1475	2075	1475	2070
y bar from Skin	(mm)	571	718	571	726
d Skin to NA	(mm)	521	693	521	704
d Web to NA	(mm)	204	332	204	319
d Flange to NA	(mm)	904	1357	904	1344
I Skin	(mm^4)	1.36E+08	4.26E+07	1.36E+08	3.33E+07
I Web	(mm^4)	1.03E+10	4.00E+10	1.03E+10	4.00E+10
I Flange	(mm^4)	1.64E+07	1.56E+07	1.64E+07	1.56E+07
Ad2 Skin	(mm^4)	4.43E+10	9.81E+10	4.43E+10	9.78E+10
Ad2 Web	(mm^4)	2.81E+09	1.32E+10	2.81E+09	1.22E+10
Ad2 Flange	(mm^4)	6.44E+10	1.38E+11	6.44E+10	1.35E+11
I Section	(mm^4)	1.22E+11	2.90E+11	1.22E+11	2.85E+11
S Flange	(mm^3)	1.31E+08	2.10E+08	1.31E+08	2.09E+08
S Skin	(mm^3)	2.13E+08	4.03E+08	2.13E+08	3.93E+08
Maximum Moment	(N-mm)	1.00E+10	3.60E+10	1.00E+10	3.60E+10
Max Stress Flange	(MPa)	76	172	76	173
Max Stress Skin	(MPa)	47	89	47	92
		Assume Segment 3	Segment 3	Segment 4	Segment 4
Elevation of Bottom of Tower Segment with Buffer	(m)	40.000	40.000	55.000	55.000
Governing MAX Vertical Steel Stress at this EL	(MPa)	425	367	426	386
Elevation of Top of Tower Segment with Buffer	(m)	55.000	55.000	71.000	71.000
Governing MAX Vertical Steel Stress at this EL	(MPa)	395	340	393	343
Elevation of Buffer Connection	(m)	55.557	52.723	63.150	60.316
Interpolated Longitudinal Skin Stress at Buffer Connection	(MPa)	393.9	344.1	409.2	371.4
Von Mises Skin Stress	(MPa)	419	396	435	425
Critical Stress In Skin (from MJK results)	(MPa)	438.0	435.0	438.0	426.0
D/C Skin Plate Stress		**96%**	**91%**	**99%**	**100%**
Check Section for Possible Web Area Reduction from Plate Slenderness					
Total Web b/t		27	66.7	27	66.7
sigma 2 / sigma 1 (Close Enough - uses Flange and Skin Stress)		-0.61	-0.52	-0.61	-0.53
k sigma		15.4	13.7	15.4	13.9
epsilon		0.715	0.715	0.715	0.715
lamda p		0.339	0.887	0.339	0.881
p (Reduction factor for plate buckling)		1	0.95	1	0.96

For now, ignore effects of local slenderness as reduction for local buckling is very small

		Sicilia - Longit Buffer	Sicilia - Transverse Buffer	Calabria - Longit Buffer	Calabria - Transverse Buffer
		Plate A - 4 meter span USING SKIN	Plate D - 8 meter span USING SKIN	Plate A - 4 meter span USING SKIN	Plate D - 8 meter span USING SKIN
Check Cutout Section for Vertical Stiffener through Plate					
Vertical Stiffener Width	(mm)	750	700	750	700
Cutout Width	(mm)	770	720	770	720
Remaining Web Width	(mm)	580	1280	580	1280
A Cutout Web	(mm^2)	29000	76800	29000	76800
d Cutout Web to NA	(mm)	589.1	692.0	589.1	678.6
I Cutout Web	(mm^4)	8.13E+08	1.05E+10	8.13E+08	1.05E+10
Ad2 Cutout Web	(mm^4)	1.01E+10	3.68E+10	1.01E+10	3.54E+10
I Total Section with Cutout Web	(mm^4)	1.20E+11	2.84E+11	1.20E+11	2.79E+11
S Flange with Cutout Web	(mm^3)	1.29E+08	2.05E+08	1.29E+08	2.04E+08
S Skin with Cutout Web	(mm^3)	2.10E+08	3.95E+08	2.10E+08	3.84E+08
Max Stress Flange (with Cutout Web)	(MPa)	78	175	78	177
Max Stress Skin (with Cutout Web)	(MPa)	48	91	48	94
Von Mises Skin Stress	(MPa)	420	398	435	426
Critical Stress In Skin (from MJK results)	(MPa)	438.0	435.0	438.0	426.0
D/C Skin Plate Stress		**96%**	**91%**	**99%**	**100%**
Check Shear Demand on Welds Between Webs and Skin, Webs and Flange					
Q Skin	(mm^3)	84967820	141617197	84967820	138995421
Q Flange	(mm^3)	71194211	101775845	71194211	100767470
Applied Shear	(kN)	20000	10000	20000	10000
VQ/I - Skin (Demand on all welds at interface combined)	(kN/mm)	13.9	4.9	13.9	4.9
VQ/I - Flange (Demand on all welds at interface combined)	(kN/mm)	11.7	3.5	11.7	3.5
Check Shear Capacity of Welds Between Webs and Skin					
Number of Fillet Welds at Interface		4	4	4	4
Effective Throat Size per Weld	(mm)	15	6	15	6
Fu Base Metal	(MPa)	540	540	540	540
Beta w		1.00	1.00	1.00	1.00
Gamma M2		1.25	1.25	1.25	1.25
Capacity per Weld	(kN/mm)	3.74	1.50	3.74	1.50
Total Weld Capacity	(kN/mm)	15.0	6.0	15.0	6.0
D/C Welds		93%	82%	93%	81%
Check Shear Capacity of Welds Between Webs and Flange					
Number of Fillet Welds at Interface		2	2	2	2
Effective Throat Size per Weld	(mm)	24	12	24	10
Fu Base Metal	(MPa)	540	540	540	540
Beta w		1.00	1.00	1.00	1.00
Gamma M2		1.25	1.25	1.25	1.25
Capacity per Weld	(kN/mm)	5.99	2.99	5.99	2.49
Total Weld Capacity	(kN/mm)	12.0	6.0	12.0	5.0
D/C Welds		98%	59%	98%	71%
Check Shear Capacity of Welds Between Vertical Plates 50 THK and Flange					
Applied Vertical Force per Vertical Plate 50 THK	(kN)	2000	1000	2000	1000
Distance to CG of Torsion Box	(mm)	1920	2200	1920	2200
Applied Torsion from This Vertical Force	(kN-m)	3840	2200	3840	2200
Required Resistance in Shear in Flange and Skin to Resist Torsion	(kN)	2695	1073	2695	1074
Height of Vertical Plate 50 THK	(mm)	3000	1500	3000	1500
Applied Force on Welds Between Plate 50 and Flange	(kN/mm)	0.90	0.72	0.90	0.72
Number of Fillet Welds at Interface		1	1	1	1
Effective Throat Size per Weld	(mm)	6	6	6	6
Fu Base Metal	(MPa)	540	540	540	540
Beta w		1.00	1.00	1.00	1.00
Gamma M2		1.25	1.25	1.25	1.25
Capacity per Weld	(kN/mm)	1.50	1.50	1.50	1.50
Total Weld Capacity	(kN/mm)	1.5	1.5	1.5	1.5
D/C Welds		60%	48%	60%	48%

5.3 Tower Leg Segment Detailed Finite Element Analysis

5.3.1 Introduction

The tower leg longitudinal plates and stiffeners are proportioned using the equivalent width method described in EN 1993-1-5 Sections 4. However, the size and complexity of the Messina Strait Bridge and the significant steel quantities in the towers warrant the verification of the resulting design with a more sophisticated detailed finite element analysis of a full tower leg segment. The objective of the detailed analysis is to confirm that design provides a sufficient safety level, without excess conservatism.

5.3.2 Finite Element Model Description

The Sicilia tower leg Segment 6 was modelled because it has thinner plates relative to the imposed axial load than other segments, and so might show greater sensitivity to buckling effects. The modelled plate thicknesses and longitudinal stiffeners are listed in Table 5-1.

Plate	A	B	C	D	E	F	G	H
Thickness	85	70	45	40	55	35	35	40
Longitudinal Stiffener	750 x 75	675 x 68	625 x 63	625 x 63	650 x 65	500 x 50	450 x 45	475 x 48

Table 5-1: Modelled plate thicknesses and longitudinal stiffener dimensions.

The analysis model differed from the final tower configuration as shown in Table 5-2. In all cases the analysis model comprised more slender elements than are present in the final configuration and so final configuration will be no more sensitive to buckling effects.

Parameter	Modelled	Current
Plate A thickness	85 mm	95 mm
Plate B thickness	70 mm	75 mm
Plate G longitudinal stiffener	450 x 45 mm	475 x 48 mm

Table 5-2: Difference between analysis model and final tower confirguration.

The model is 18 metres high and comprises six 3 m spans of longitudinal stiffeners. Both the plates and the vertical stiffeners are modelled with shell elements. Bar elements are used to model the horizontal stiffeners and to represent the restraint provided by triangular diaphragms. Bar elements simplify the model and are sufficient. Moments and forces are applied to the top and bottom of the model through a "spider" of rigid elements. Plan and isometric views of the model are shown in Figure 5-1.

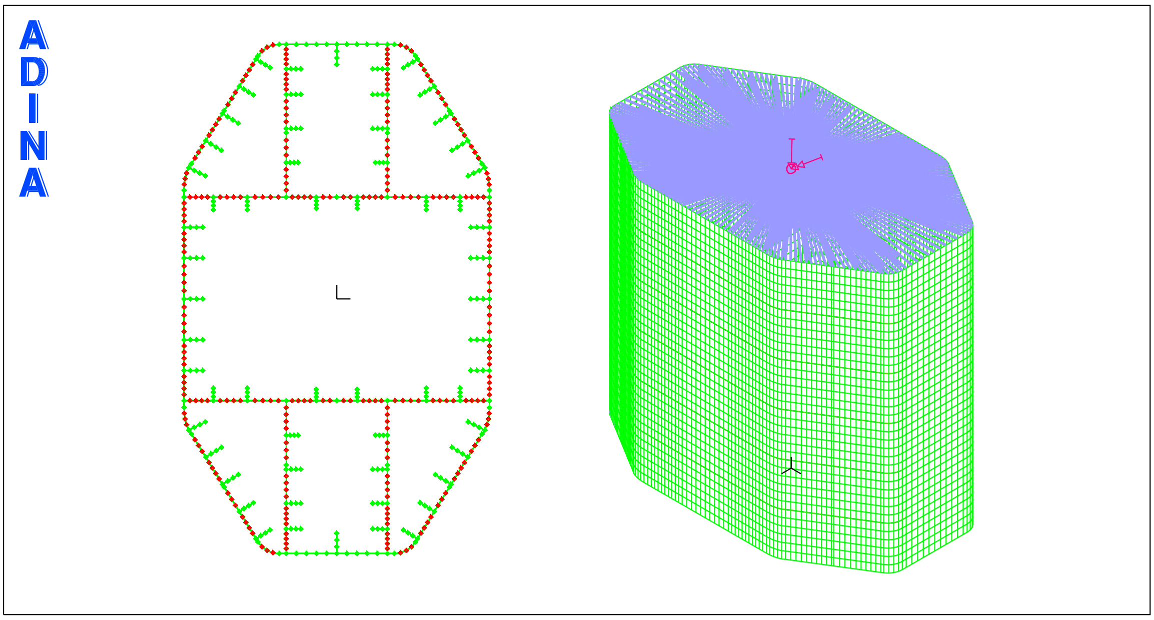


Figure 5-1: Plan and isometric views of model meshing.

It was originally planned to include the effects of imperfections by modelling the plates and stiffeners with the equivalent imperfections specified in EN 1993-1-5 Annex C, using the geometrical form of the elastic buckling modes of the panels as the basis. Several issues were encountered with this approach:

- The 1:400 "equivalent imperfection" specified for longitudinal stiffeners in EN 1993-1-5 Table C.2 is no greater than the manufacturing tolerance allowed in EN 1090-2 Table D.1.6. This means that if the tower leg panels were manufactured to the allowable tolerances, the analysis would have been made with no allowance for residual stresses and therefore might be unconservative;
- The "equivalent imperfections" specified for a plate between stiffeners in Table C.2 will cause a reduction in the resistance of the plate, whereas EN 1993-1-5 Section 4 gives no reduction for the plate slenderness for the width-to-thickness ratios used in the majority of panels in the tower legs;

- It is very difficult to find the most onerous arrangement of “equivalent imperfections” for stiffener twist because there are so many different combinations possible; and
- There is no single buckling mode in the panels that gives magnitudes of out-of-plane panel deformations similar to those specified in EN 1993-1-5 Table C.2, so an alternative method of generating suitable initial imperfections was required.

It was concluded that the most realistic resistance assessment would be provided by:

- Using out-of-plane geometrical imperfections along the longitudinal stiffeners in the form that occurs when applying equal line loads along each stiffener and accept the resulting imperfections in the plate between the stiffeners and twist of the stiffeners as the appropriate values for those initial imperfections.The initial imperfections are proportional to deformations from applied line loads. These line loads are applied along the line of the vertical stiffeners and in the plane of the stiffener (perpendicular to the skin plate). The line loads on adjacent panels act in opposite directions so as to produce a deformed shape that is alternately into and out from the centre of the leg in alternate panels. The applied loads themselves are uniformly distributed. The maximum value of geometrical imperfection in each panel was taken as the allowable fabrication tolerance in the stiffeners of 1:400 as EN 1090-2 Table D.1.6.
- Accounting for residual stresses directly (not by equivalent imperfections) by using an appropriate stress-strain curve. The residual stresses were incorporated by modifying the stress-strain curve from bi-linear elastic/plastic to multi-linear to represent the average stress-strain response of steel with residual stresses and including the 0.2% proof strain that is expected with higher strength steels such as S460.To give an assessment of the magnitude of these effects, the three cases of maximum axial with coexistent moments, and maximum moments with coexistent axial and moment were analyzed using bi-linear elastic/plastic material properties. These showed resistances only slightly higher than with residual stresses, with 2% greater for the load combination with maximum longitudinal moment and 4% for the load combination with maximum axial compression. However, the rotations were of the order of 50% less than in the analyses using residual stresses.

5.3.3 Details of the Finite Element Model

The FE model solution is non-linear considering large strains and displacements and non-linear material properties. The model has approximately 22,500 nodes and 22,600 shell and beam members. The FE program used is ADINA version 8.4.

The model uses four node shell elements for the plates and longitudinal stiffeners. The mesh spacing length in the vertical direction is 0.5 m. The meshing of the plates is typically five spaces between longitudinal stiffeners and three spaces through the depth of the longitudinal stiffeners. The transverse stiffeners, other than the top and bottom diaphragms, are modelled as equivalent frames of beam elements of equivalent area and moment of inertia located along the shell nodes. These elements have elastic material properties. Beam elements are used to simplify the modelling and reduce the computation time. Because the initial imperfections are applied so that there are node lines at the transverse stiffeners and because these stiffeners have sufficient stiffness to retain these nodal lines, the behaviour of the model is insensitive to the representation of the transverse stiffeners and this makes beam elements appropriate. Shells with different thickness are aligned along the shell centreline rather than the outside face. This was done to avoid the modelling problems of modelling small offsets which either may create unrealistic local moments when modelled as rigid links or constraint equations or may require undesirable element geometry if modelled with small elements forming the step in the plate centre-lines. When using shell elements, whatever modelling technique is used, there is some discrepancy between the model and the reality. However, the discrepancy will have a negligible effect on the results because, relative to the section size, the influence of the eccentricity between two plates of different thicknesses is not important. One advantage of adopting the centreline alignment is that it uses the minimum number of nodes and thus keeps the structure stiffness matrix to the minimum size, which is an advantage in non-linear analysis because of the storage size and computation time required for each iteration.

In ADINA, the model axes are X for the transverse direction, Y for the longitudinal direction, and Z for the vertical direction. In IBDAS, the model axes are Y for the transverse direction, Z for the longitudinal direction, and S (equivalent to X) for the vertical direction. The inclination of the tower leg is not modelled, as it is not significant for the purposes of this analysis. The vertical distance between the top and bottom planes is 18 m. The maximum dimension along the bridge longitudinal axis is 20 m and the maximum dimension along the bridge transverse axis is about 12 m. The bolted splice 1m above the bottom diaphragm is not modelled because the goal is to compare the finite element results with those of EN 1993-1-5 Section 4.

The section resistance is compared with the resistance for the same section calculated using EN 1993-1-5 Section 4. The thickness changes in the design process using the effective width method and in the finite element model are made at the plate intersections, and so the two methods are comparable.

Units are in m, MPa, and MN, unless noted otherwise.

5.3.4 Application of Loads

The loads are applied at each segment end by a "spider" of rigid links radiating from a node at the centre of the cross-section to the nodes at the ends of the skin-plates and longitudinal stiffeners. The top node is the load application point, and the bottom node is the support point at which all six degrees of freedom are restrained.

Ideally, the forces and moments would be applied to the model at the centroid of the stiffened elements without either applying local moments or applying local constraints. However, this is not achievable. For example, to avoid moment constraint in the plane of the stiffener, the forces should be applied at the centroid of the stiffener, but the centroid is not at the centroid of the skin-plate, so application at the centroid requires an overload of the stiffener. The method of application through rigid connection to the nodes of both the skin and the stiffeners avoids overloads while creating some constraint. However, the segment model comprises six panels between transverse stiffeners and the inner four panels, especially the inmost two, are so far from the constraints that under failure governed by plasticity (as in this structure) the constraints have minimal effect.

5.3.5 Material Properties

Three material models are used to describe the steel element behaviours: elastic, bilinear, and smoothed multi-linear. The elastic material model has a modulus of elasticity of 210,000 MPa and a Poisson's ratio of 0.3 as defined in EN 1993-1-1 Section 3.2.6. No thermal coefficient or material density parameters were actively used in the modelling.

The bilinear material model is similar to the elastic material until yield. The yielding point is set to 460 MPa. The material modulus after yielding is set to 21 MPa, or 1/10,000th of the elastic value. This is done to avoid potential numerical integration problems.

As described in Section 5.3.2, the "equivalent imperfection" specified in EN 1993-1-5 Table C.2 at 1:400 for longitudinal stiffeners is no greater than the manufacturing tolerance allowed in EN 1090-2 Table D.1.6. This means that if the panels were manufactured to the allowable tolerances, the analysis would have been made with no allowance for residual stresses and therefore might be un-conservative. Therefore, the maximum value of geometrical imperfection in each panel was taken as the allowable fabrication tolerance in the stiffeners of 1:400 as in EN 1090-2 and the residual stresses were incorporated by modifying the stress-strain curve from bi-linear elastic/plastic to multi-linear to represent the average stress-strain response of steel with residual stresses. The multi-linear material is based on the bilinear material model, but modified to incorporate an initial residual stress curve and the 0.2% proof strain expected with a higher strength steel such as the S460 used in the tower. In the absence of better information, the initial residual stress curve was taken from:

M. B. Prime, Los Alamos National Laboratory, "Residual stresses measured in quenched HSLA-100 steel plate," Proceedings of the 2005 SEM Annual Conference.

The steel has a yield stress of 690MPa and is quenched and tempered. The plate thickness was 61 mm, so similar to many plates in the tower. This is stronger than the Messina steel grade and is manufactured by a different process, but the Eurocode part for high strength steels, EN 1993-1-12 Section 2.1 "Additional rules to EN 1993-1-1" says that Table 6.2, Selection of buckling curves, should use the same buckling curves (so the same effects of imperfections and residual stresses) for high strength steels as for S460 steel. The residual stress pattern from the above paper is given by: $S/Sy = 0.29 - 0.96y^2 - 3.371y^4 + 11.568y^6 - 12.179y^8 + 4.455y^{10}$, and is shown in Figure 5-2, where *S* is the initial residual stress, *Sy* is the yield stress, and *y* is the normalized through-thickness position or relative depth, which ranges from -1 to +1, so the total relative thickness is 2. The maximum residual stresses, from the curve described above, are about 24% and 29% of yield for compression and tension, respectively. Given the uncertainty of these values for S460 material, the residual stresses from welding and cutting activities were not added to the residual stresses above because the apparent improvement of accuracy from is likely to be an illusion. These other residual stresses might be of the order of 2% to 5% of yield compared with the residual stresses in the plate that are 25% to 30% of yield. Higher residual stresses would reduce the cross section capacity.

The proof stress is assumed to begin at 85% of the yield stress. The proof strain is assumed to be an additional 0.2% for a total of about 0.00419. The stress-strain curves considered are shown in Figure 5-3. For all cases, the rupture strain, or upper limit strain was set to 10, so a rupture scenario is not considered.

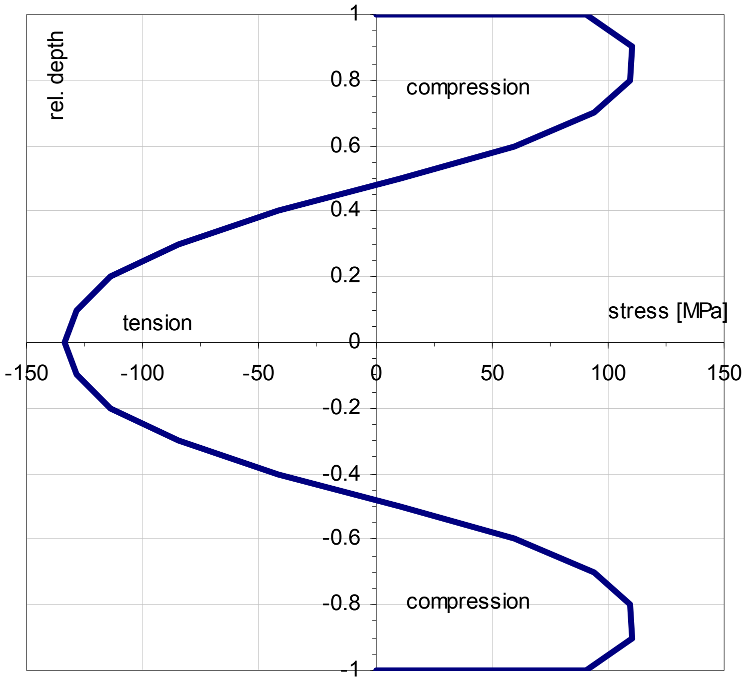


Figure 5-2: Initial residual stresses vs relative through-thickness.

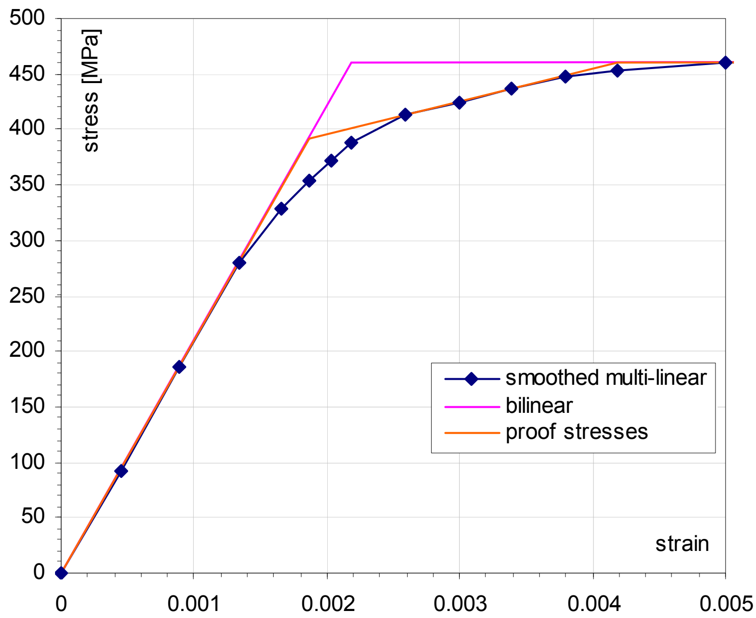


Figure 5-3: Stress – strain relationships for different material models.

5.3.6 Run cases

Three types of run cases were used:

1 The imperfection load case comprising patterns of line loads;

2 The multi-linear case; and

3 The bilinear material case;

The deformed geometry of the FE model of the imperfection case was used as the initial geometry for the bilinear and multi-linear cases.

5.3.6.1 Imperfection case

The type of imperfection modelled is from EN 1993-1-5 Annex C Figure C.1 “global longitudinal stiffener”. From EN 1993-1-5 Annex C Table C.2, the imperfection magnitude is the minimum of a/400 and b/400, so the dimension used to calculate the magnitude of the imperfection is 3000 mm (the spacing of the transverse stiffeners) in all plates except plate A, for which the governing dimension is the plate width of 4000 mm. Therefore, the imperfection values introduced were ±10 mm for the plate A stiffener, and ±7.5 mm for all other stiffeners.

The use of buckling mode shapes as a source of model imperfections was attempted. However, as noted above, it was found that there was no single buckling mode that gave similar magnitudes of out-of-plane deformation of the panels, so an alternative method of generating suitable initial imperfections was required.

The imperfect shape was generated by applying lateral line loads along the longitudinal stiffeners. The magnitude of line loading was iterated until the amount of desired imperfection was obtained. The corner or plate intersection regions were restrained from movement in the horizontal plane to prevent additional deformations in the model. In addition, the line loading was applied on the stiffeners symmetrically in plan. The line loads were applied such that the stiffeners would deform in opposite directions in adjacent panels. The deformed node coordinates were saved and used as initial coordinates for the remaining run cases. The model deformed with the initial imperfections is shown in Figure 5-4. The region of interest is mainly the plates between the second and fourth transverse stiffener (two inner most panels).

Realistic twist imperfections in the longitudinal stiffeners were not modelled. However, for the width-to-thickness ratios of the stiffeners and plate thicknesses to which they are welded, realistic twist angles are unlikely to significantly affect the capacity.

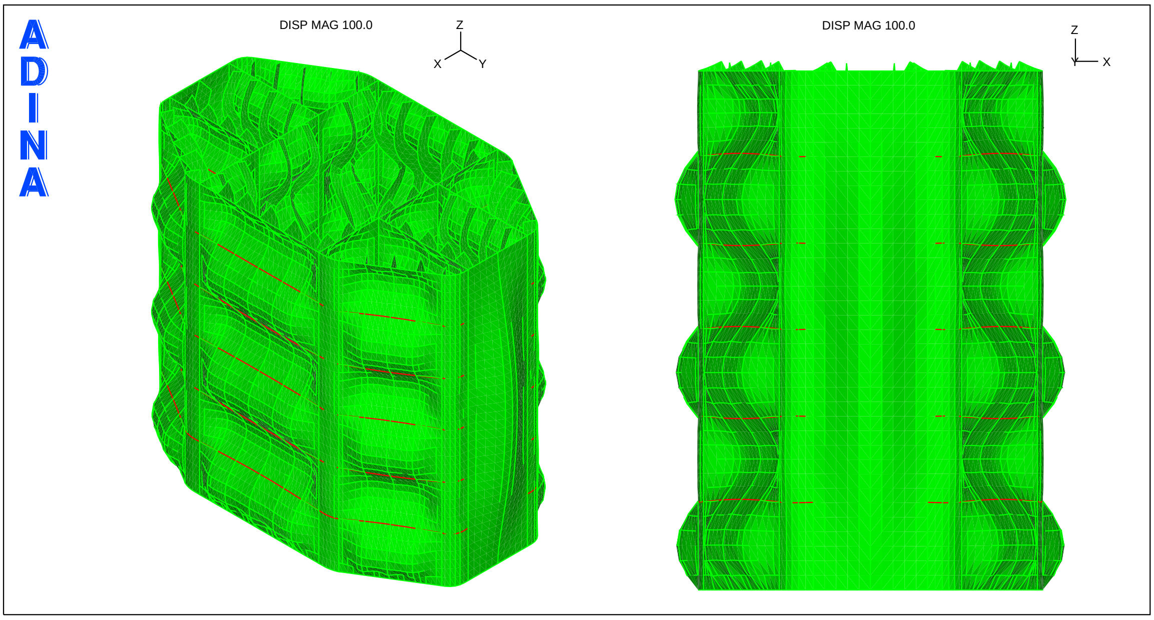


Figure 5-4: Magnified isometric and plan views of deformed model used as initial imperfections on later runs.

5.3.6.2 Multi-Linear Cases

The force and moment demands are applied to the model incrementally. ADINA is set to increment the applied load until the failure load is reach or up to 160% of the prescribed demands are applied.

The loads are from IBDAS model 3.3f and are a combination of the ULS seismic demands (Comb 5: 7 – ULS finished bridge, PP + PN + QA + VS_dyn), worst temperature loading envelope, and global imperfection loads.

The maximum seismic demands and temperature loads are added together and grouped into twelve cases: min NS, max NS, min MY, max MY, min MZ, max MZ, min VY, max VY, min L001, max L001, min L002, and max L002, where:

NS = axial load;

My = longitudinal moment (bending of the leg in a plane parallel to the bridge centreline;

Mz = lateral moment (bending of the leg in a plane perpendicular to the bridge centreline);

Vy = transverse shear (shear perpendicular to the bridge centreline;

L001 = Linear combination giving the maximum stresses at the intersection of plates A, B and E, assuming elastic behaviour and a fully effective cross-section; and

L002 = Linear combination giving the maximum stresses at the intersection of plates C, D and H assuming elastic behaviour and a fully effective cross-section.

The global tower leg imperfection loads, associated with the most severe of the global longitudinal and transverse buckling modes, are added to the demands from the IBDAS output.

The axial loads and shear forces applied at the segment top are adjusted slightly so that the reactions at the bottom (moments and axial load) are matched to the values from the combined cases, as indicated above.

RESULTS

The analysis results for the 13 load cases are presented in Figure 5-5 to Figure 5-16. Force effects, expressed as the fraction of the design load (referred to as a load factor; the proportion of

E_d in EN 1990 Section 6.3.2) applied at each load increment, are plotted on the vertical axis. The values of the design loads for each analysis case are listed in Table 5-3 with the figure in which the analysis results are plotted. Deformation (shortening and end-rotations) results, expressed as the fraction of the elastic deformations resulting from the design load applied at each load increment, are plotted on the horizontal axis. Each figure includes lines representing secant stiffnesses of 1.0, 1.5, 2.0 and 3.0 times the initial elastic stiffness.

The section resistance evident from the FE results should be at least γ_{M1} (partial factor considering buckling) times the demand to comply with the Eurocode requirements for compression members. This is confirmed by the use of γ_{M1} in EN 1993-1-1 Sections 6.3 and 5.3.2(11), and EN 1993-1-5 Section 10, which is intended for use with the output from computer models. The plots of the analysis results have a horizontal line at 1.1 on the vertical axis (load factor) indicating the load factor that must be achieved to account for the $\gamma_{M1} = 1.1$ used in design.

Responses for each of the load cases are characterized by their degree of non-linearity. The results of the 12 analysis cases can be classified into three types of response: essentially linear, moderately non-linear and highly non-linear. The essentially linear cases are max Ns, max Mz, min Vy, max L001 and max L002, shaded light yellow in Table 5-3. These cases comprise relatively low axial compressive loads and low to moderate longitudinal moments. The moderately non-linear cases are min My, min Mz and max Vy, shaded light blue in Table 5-3. These cases comprise low axial compressive loads combined with moderate to high longitudinal moments or moderate axial compressive loads combined with low longitudinal moments. The highly non-linear cases are min Ns, max My, min L001 and min L002, shaded pink in Table 5-3. These cases comprise high axial compressive loads combined with low to moderate longitudinal moments.

For the essentially linear cases, the section stiffness, axial and rotational, is maintained throughout the range of load increments up to well above the design load. For the moderately non-linear cases, the section stiffness departs from linearity but not significantly until well after the design load is attained. For the highly non-linear cases, the section stiffness deviates from linearity well below the design load and achieves a characteristic resistance only slightly higher than the required 1.1 x the design load. While part of the non-linearity is due to geometric non-linearity, which is reversible, the rest is due to irreversible plasticity, which will absorb significant energy for seismic loads approaching the section capacity.

The results of the analyses indicate that the behaviour of the section is most dependent on the magnitude of the axial compressive load and less dependent on the magnitude of the moment. The reason for this is that high uniform axial stress levels reduce the opportunity for compressive stresses to redistribute on the section as the extreme fibres approach yield due to the concurrent flexure. In the absence of high axial stresses, yielding caused by flexure is generally more localized and the yielding regions can be more effectively supported by the lesser stressed adjacent regions.

The most dramatic result of the analysis is the reduction of the section stiffness, and in particular the rotational stiffness, as the load increases. The highest utilization ratios and the greatest stiffness losses occur for the minimum linear combinations L001 and L002, for which the analysis results are plotted in Figure 5-13 and Figure 5-15. Because the leg is a tall compression member, the member stiffness has significant influence on the structure’s capacity. Therefore, the segment stiffness must be considered together with the capacity to obtain a reliable assessment of the overall effect on the tower capacity.

Case	Figure	Ns (MN)	My (MNm)	Mz (MNm)	Vy (MN)	Vz (MN)	T (MNm)
min NS	Figure 5-5	-2175	1957	-420	31	-54	22
max NS	Figure 5-6	-836	-4428	-273	-2	16	-37
min MY	Figure 5-7	-1009	-7373	-458	18	38	2
max MY	Figure 5-8	-1995	4918	-226	1	-24	-3
min MZ	Figure 5-9	-1730	-1753	-1034	20	-203	-104
max MZ	Figure 5-10	-1368	-2305	954	-17	42	77
min VY	Figure 5-11	-1536	-990	798	-26	6	98
max VY	Figure 5-12	-1620	-2820	-786	23	7	-126
min L001	Figure 5-13	-2105	4809	-203	22	-19	-3
max L001	Figure 5-14	-913	-7265	-372	7	33	-1
min L002	Figure 5-15	-2119	4382	404	-6	-27	-7
max L002	Figure 5-16	-894	-6637	-649	21	58	-2

Table 5-3: Design loads for each analysis case (loads at the top of the segment).

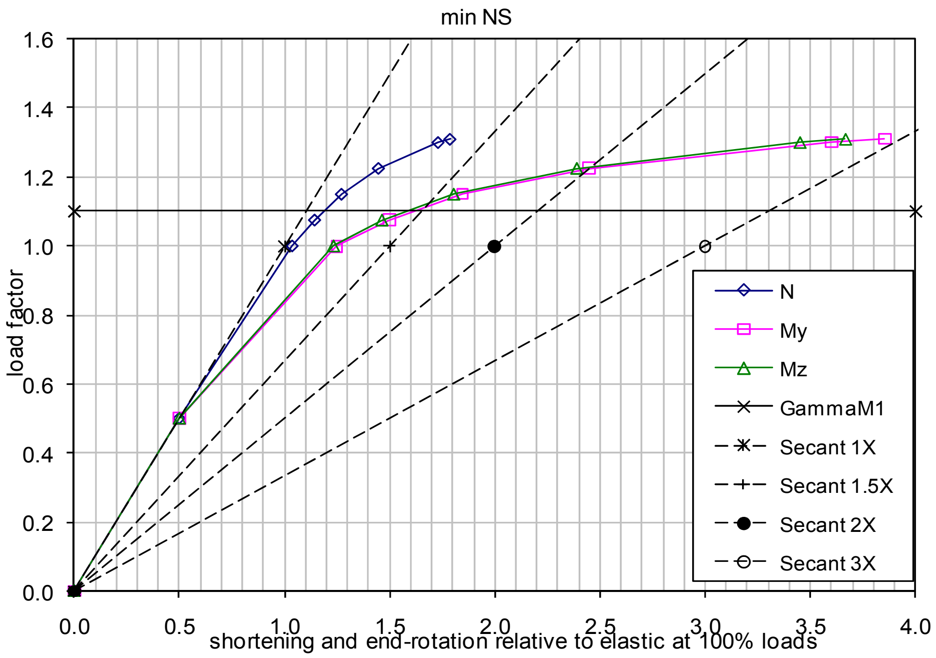


Figure 5-5: Minimum axial (Ns).

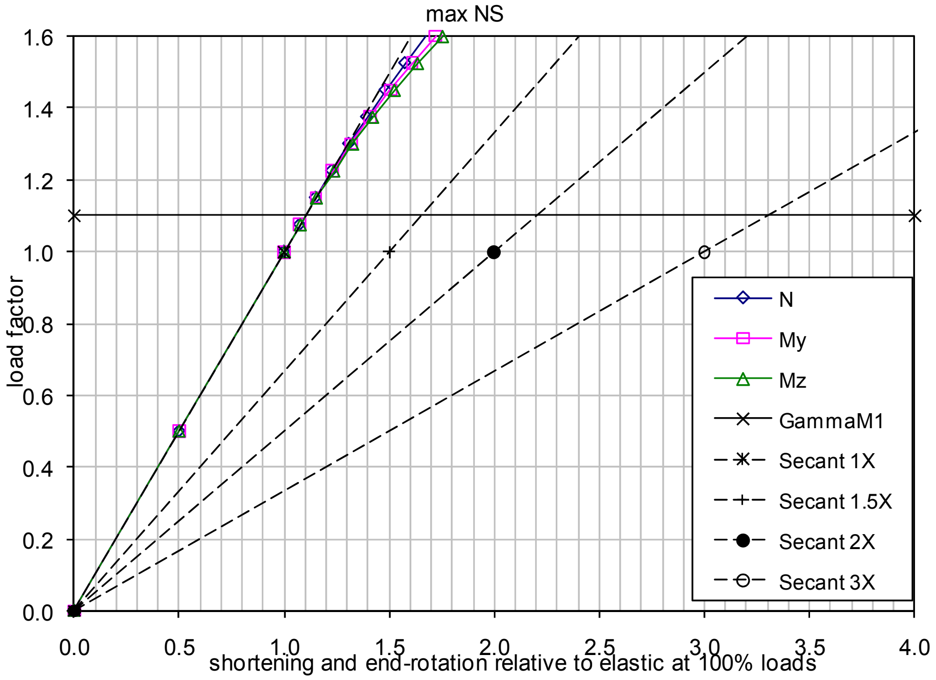


Figure 5-6: Maximum axial (Ns).

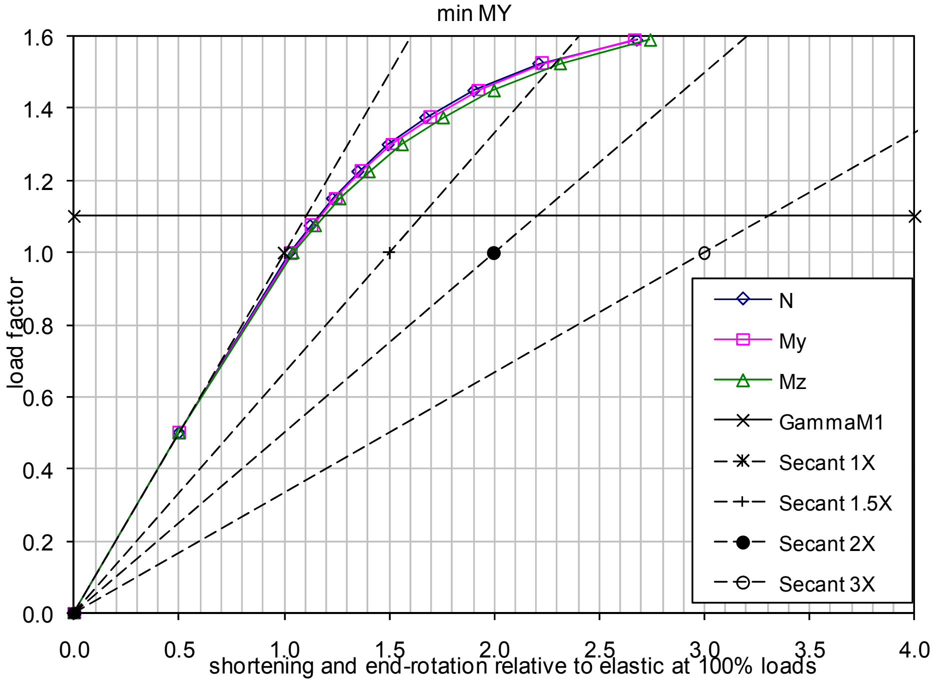


Figure 5-7: Minimum longitudinal moment (My).

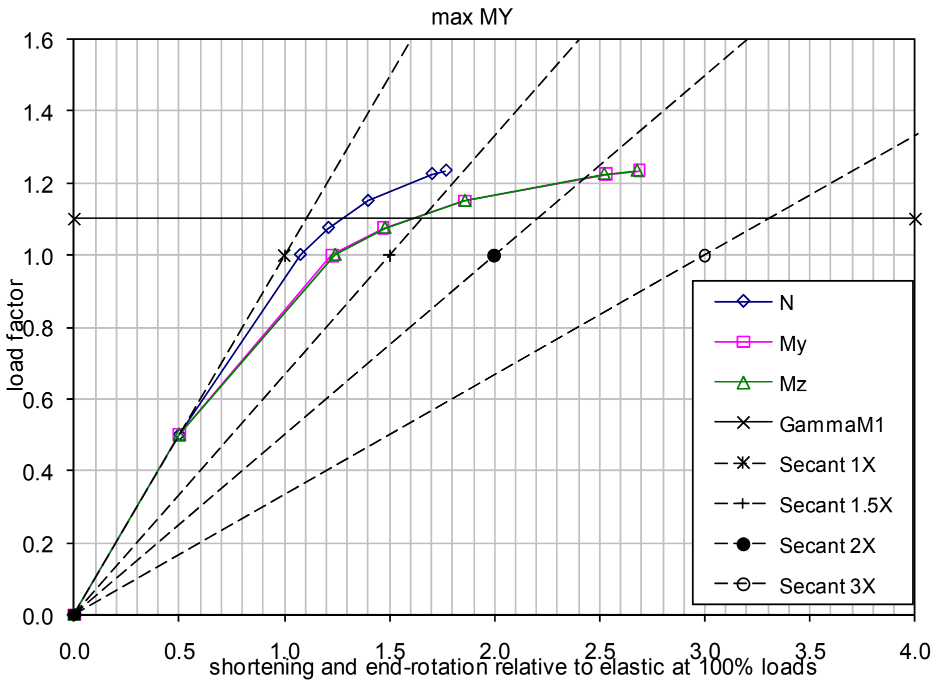


Figure 5-8: Maximum longitudinal moment (My).

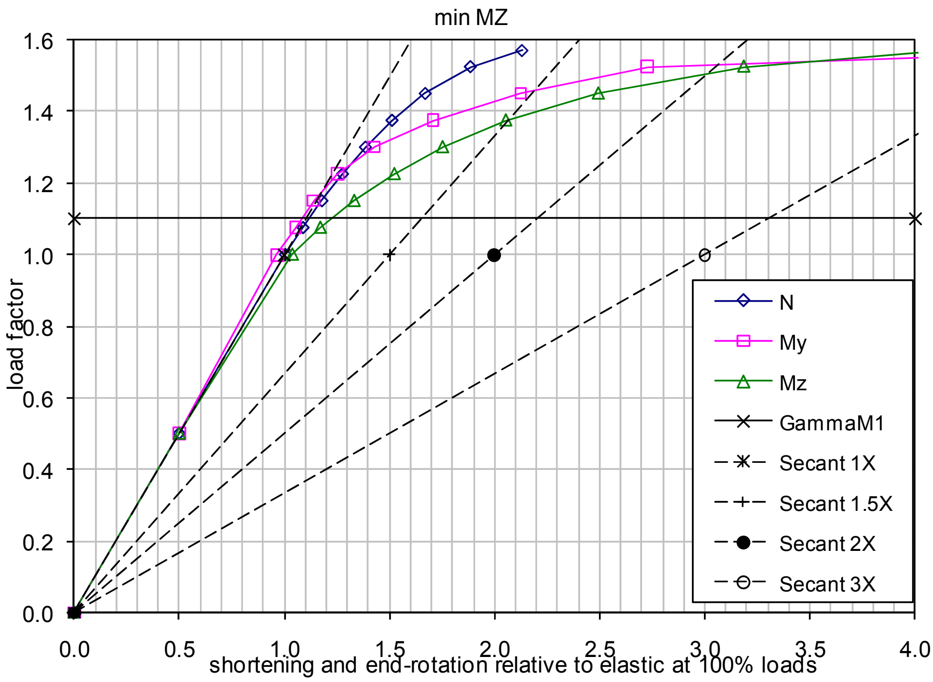


Figure 5-9: Minimum transverse moment (Mz).

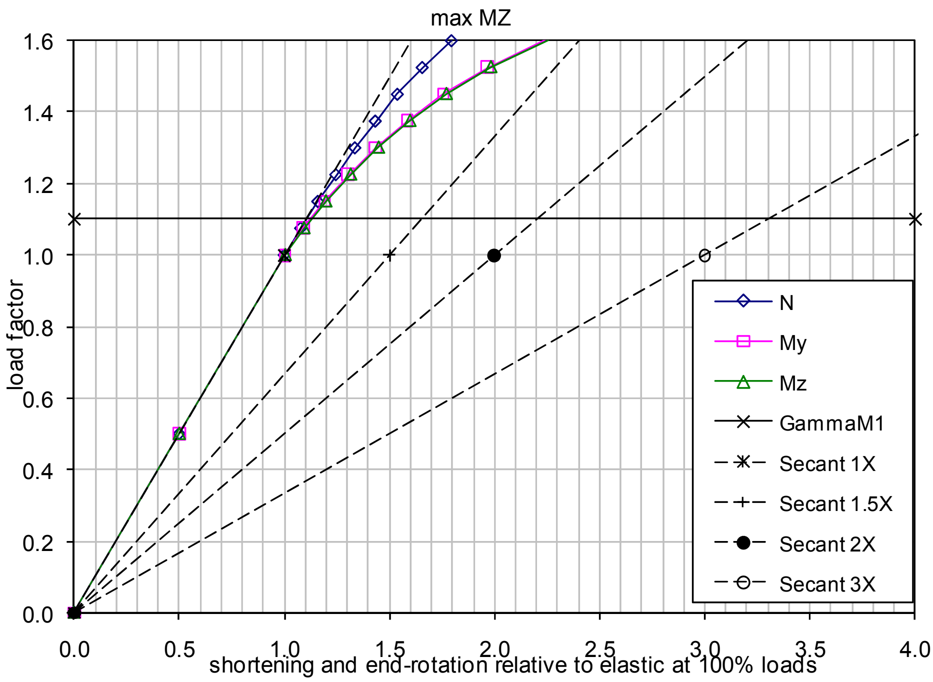


Figure 5-10: Maximum transverse moment (Mz).

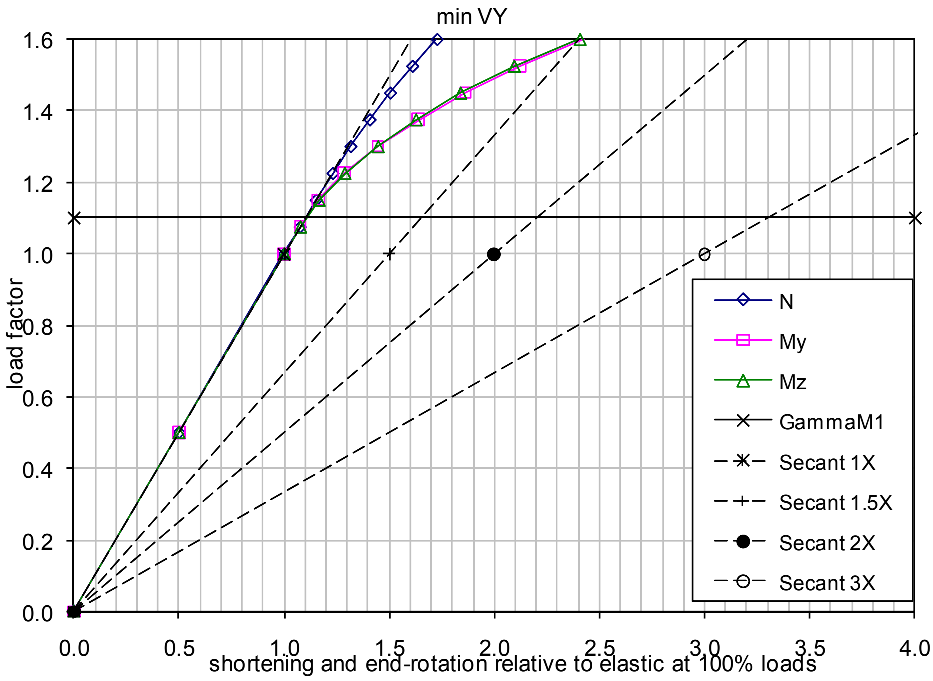


Figure 5-11: Minimum transverse shear (Vy).

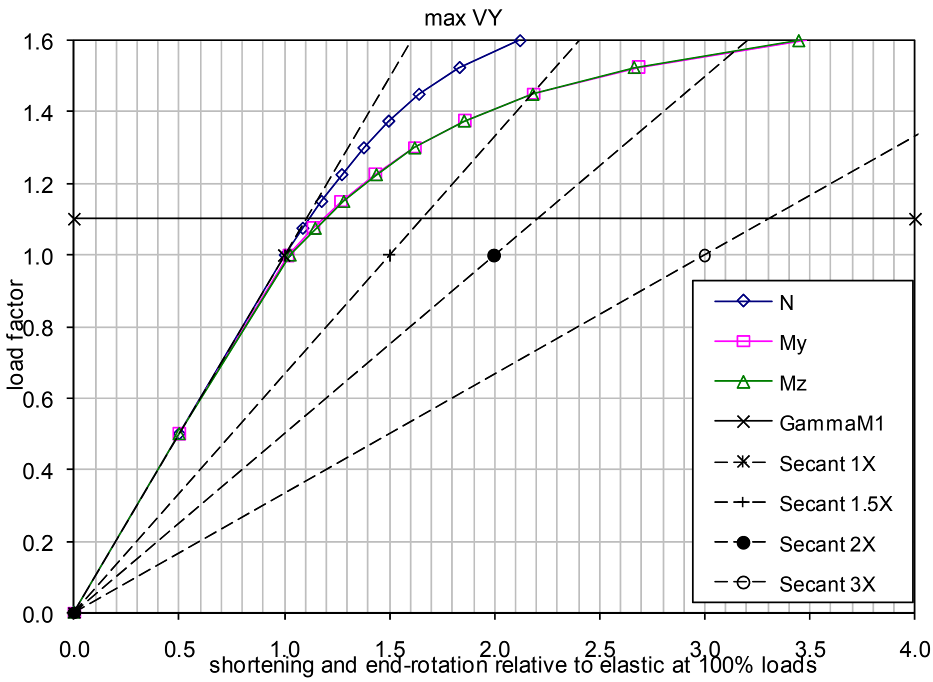


Figure 5-12: Maximum transverse shear (Vy).

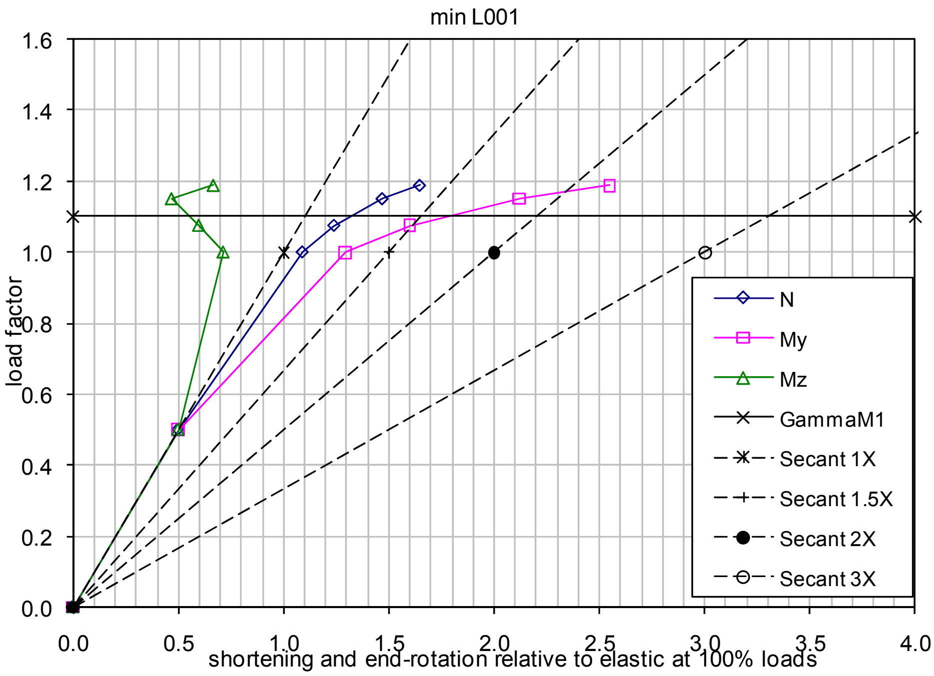


Figure 5-13: Minimum linear combination L001.

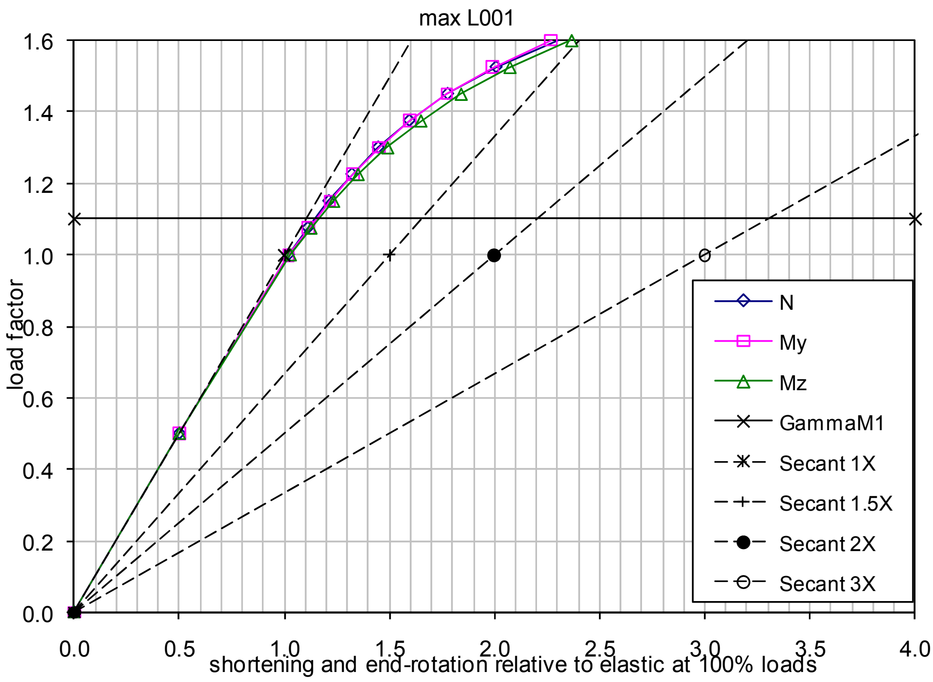


Figure 5-14: Maximum linear combination L001.

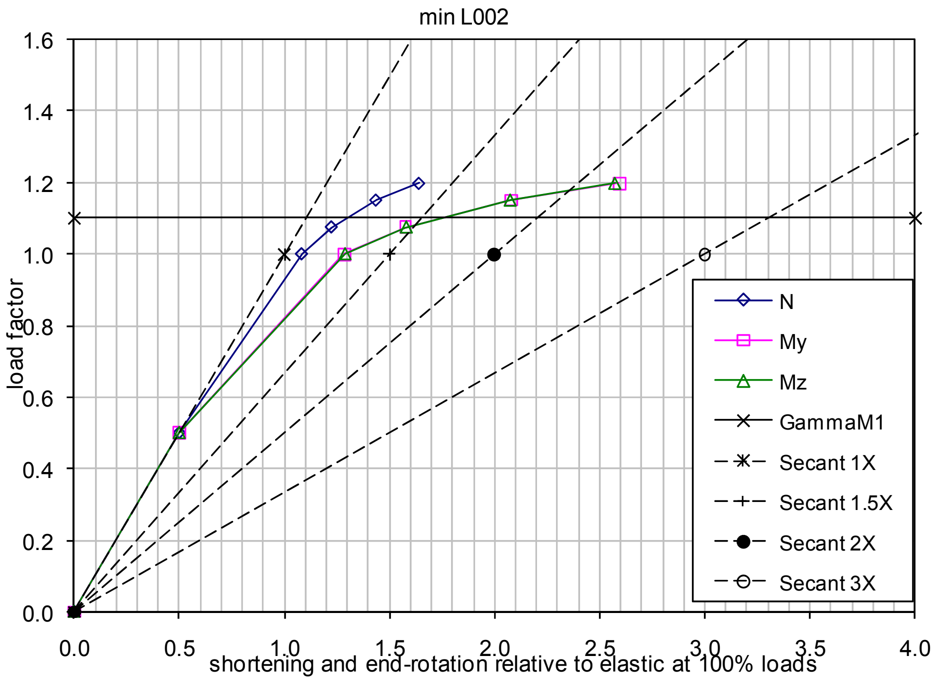


Figure 5-15: Minimum linear combination L002.

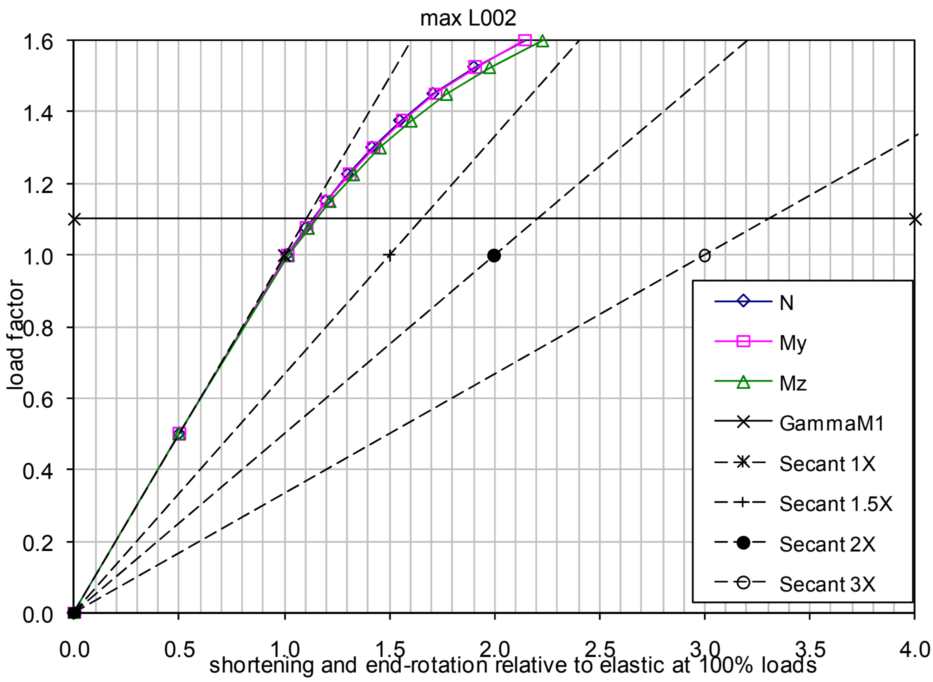


Figure 5-16: Maximum linear combination L002.

<u>Stiffness</u>

The loss of stiffness is shown by the flattening of the load response curves in the figures above. The flexural secant stiffness has decreased significantly at the failure load in many cases. This increases the second-order effects and therefore increases the bending moments above the design values. For example, for min L002, shown in Figure 5-15, the secant stiffness has decreased at failure by a factor of 2.17, (i.e. the secant stiffness has reduced to 46% of the elastic stiffness).

<u>Strength</u>

For the most onerous loading with the greatest reduction in secant stiffness, as shown in Figure 5-15, the maximum cross-sectional resistance reaches 1.197/1.1 = 1.09 times the required resistance based on the design values of forces and moments. However, at this loading the secant stiffness has dropped so much that the actual moments are greater than the design values, so the effective increase in strength is less. The increased total moments were calculated by separating them into moments that are increased by reduced secant stiffness (moments from seismic action and buckling loads) and loads that are reduced by reduced secant stiffness (moments from restraint of the tower top by the cables). These moments were then factored to allow for the reduced secant modulus, which gave a modest increase in the total moment.

The resistance of the section with these greater moments was calculated using the average strain at the centerline of plate A as the governing criterion. The average strain was calculated as:

{shortening + [end rotation × distance from centre of section to plate A]}/height

At the maximum load sustained by the FE model, this strain was -3.75×10^{-3}. The strain arising from the increased moments (due to reduced secant stiffness) was calculated from the ratio of the increased moments to the moments applied to the FE model at each load factor. These increased strains are shown in the Figure 5-17. The maximum average strain reaches the limiting value at a load factor of approximately1.17. Therefore the maximum design resistance = 1.17/1.1 = 1.064 times the required resistance based on the design values of forces and moments, giving a utilization ratio of 0.94.

Using EN 1993-1-5 Section 4, the legs were designed in this segment for a maximum utilization ratio in the range of0.99. Therefore, the FE predicts a resistance of 0.99/0.94 = 1.05 times that predicted by EN 1993-1-5 Section 4, an increase in resistance of 5%.

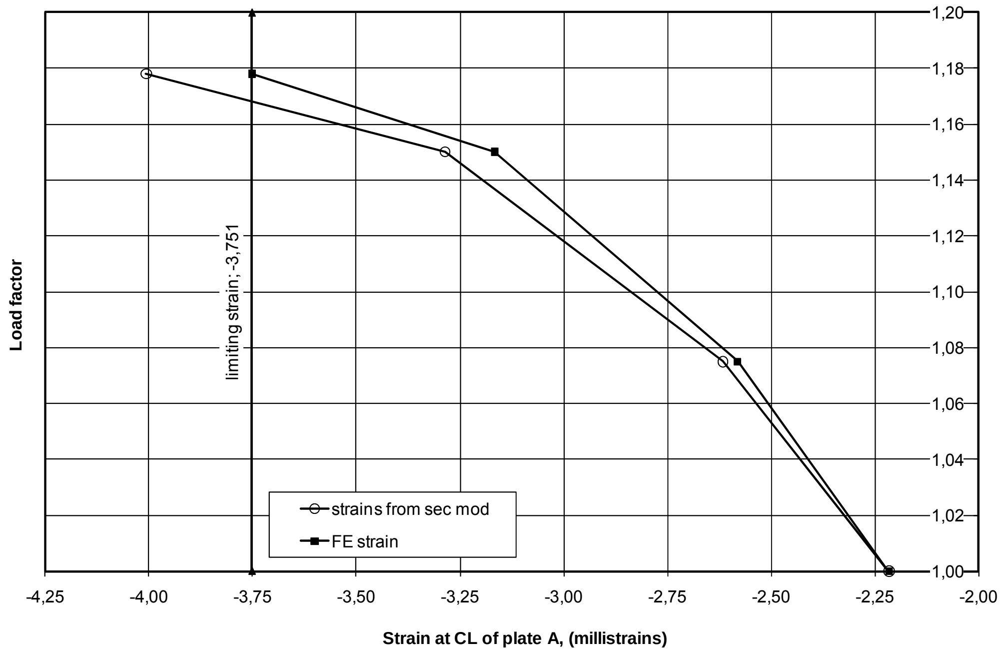


Figure 5-17: Calculated total strain compared with limiting strain (strain at failure).

5.3.6.3 Bilinear Cases (Effect of residual stresses)

To give an assessment of the magnitude of the effects of residual stresses, the three cases of maximum axial load with coexistent moments, maximum longitudinal moment with coexistent axial load and transverse moment and maximum transverse moment with coexistent axial load and longitudinal moment were analyzed using bi-linear elastic/plastic material properties. This assessment was done using a previous iteration of the tower leg design, for which the plate thicknesses were typically less than those present in the current design.

The results of these analyses are shown Figure 5-18, Figure 5-19 and Figure 5-20, respectively. In each plot, the load deformations curves for the analyses with residual stresses are also shown. In the caption for each figure the ratio of the characteristic resistance, R_k, for runs with and with residual stresses, to the design load, E_d, are noted.

The analyses without residual stresses showed resistances only slightly higher than with residual stresses. Without residual stresses, the resistance is 2% greater for the load combination with maximum longitudinal moment and 4% greater for the load combination with maximum axial compression. The rotations at failure were of the order of 50% less than in the analyses using residual stresses.

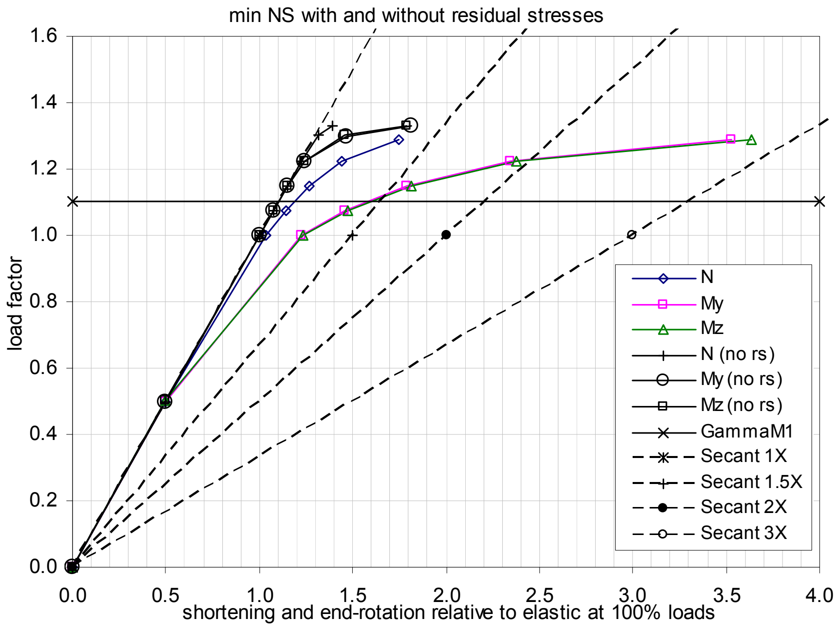


Figure 5-18: Minimum axial, comparison of material properties (Characteristic resistances, R_k of 129% of E_d vs. 133% of E_d).

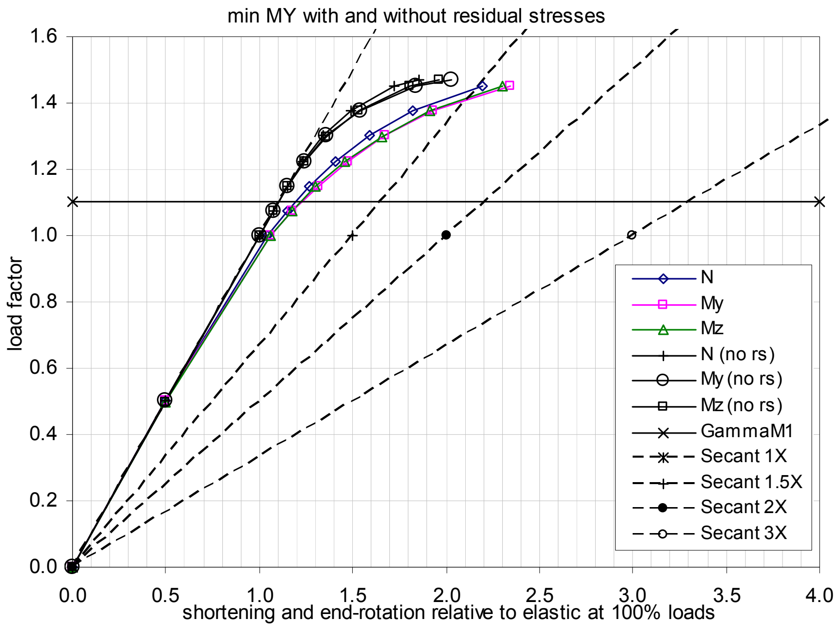


Figure 5-19: Minimum longitudinal moment, comparison of material properties (Characteristic resistances, R_k of 145% of E_d vs. 147% of E_d).

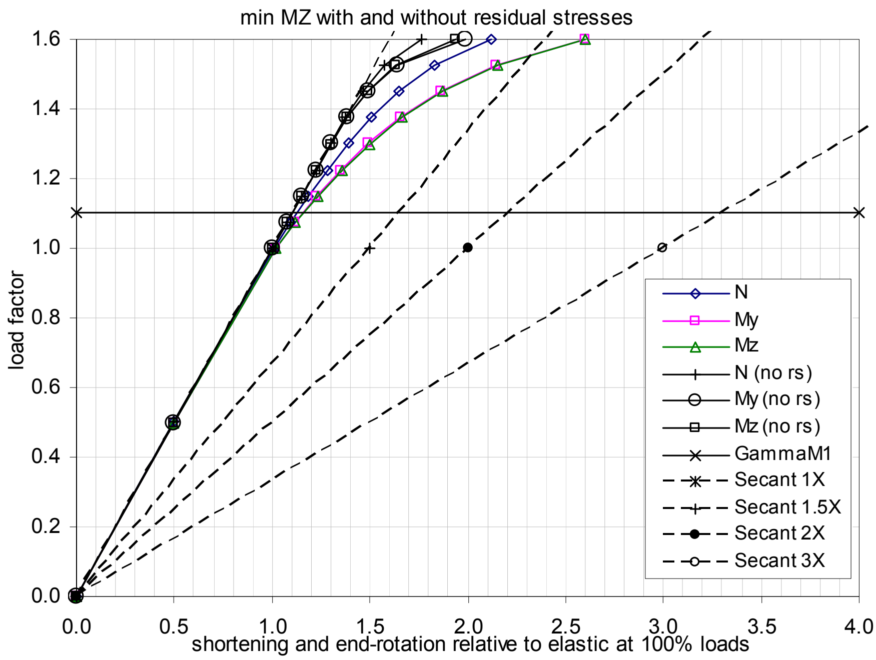


Figure 5-20: Minimum transverse moment, comparison of material properties.

5.3.6.4 Effect of Model Restraint to the Plate A Longitudinal Stiffener

The introduction of the loads into the model is made by a "spider" of rigid elements connected to the nodes in the end planes of the model. The chosen arrangement of the rigid elements avoids local overloading of the structure, but causes an artificial restraint against local rotation of the plates and stiffeners at the ends of the model. This has an insignificant effect in general because the model comprises six panels in height, so the central bays are remote from the restraint. The one element that might show sensitivity to the restraint from the spider is the plate A longitudinal stiffener. This stiffener does not rely on frequent transverse stiffeners for stability and has a buckling length of approximately 10 m. The effects of these artificial restraints was determined using the earlier version of the model. The load case producing the highest demands on plate A was run both with the rigid links connecting to the plate A longitudinal stiffener and with the rigid links removed. The results of this analysis are shown in Figure 5-21. The characteristic resistance was 117.6% of the design load with the single point connection compared with 117.8% of the design load for the connection to all the nodes, showing an insignificant reduction in resistance of the whole cross-section.

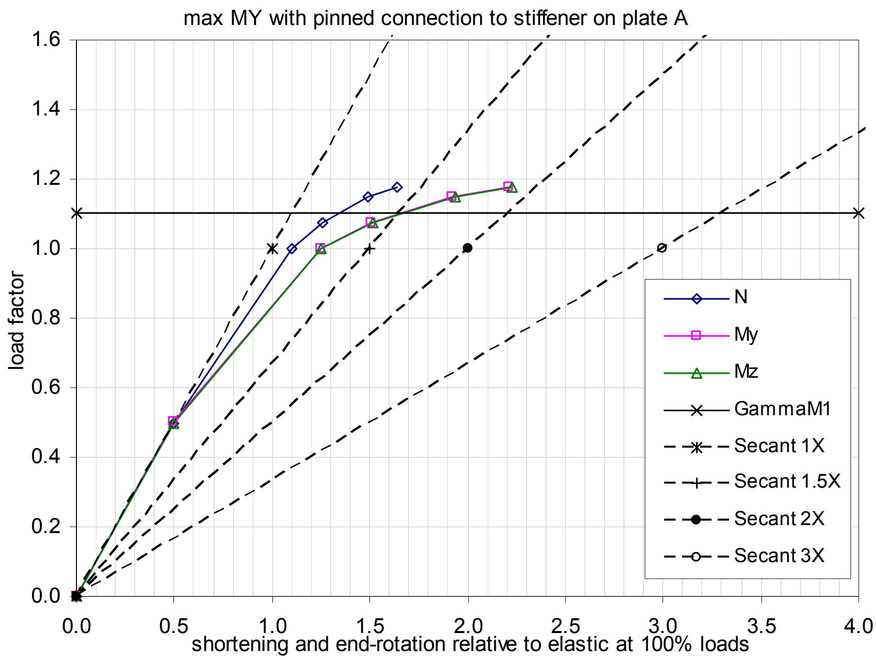


Figure 5-21: Maximum longitudinal moment with single point connection to plate A stiffener.

5.3.7 Conclusions

- The resistance of Segment 6 from the finite element model is 5% greater than the resistance calculated by EN 1993-1-5 Section 4;
- The section response is most dependent on the magnitude of the axial compressive stress;
- The level of safety in the tower as proportioned using the equivalent width method is appropriate, providing a reasonable balance of reliability and economy; and
- The portion of the non-linear behavior attributable to plasticity will absorb significant energy for seismic loads approaching the section capacity.

5.3.8 Assessment for the Envelope of Seismic Time-History Analysis Force Effects

A second detailed tower leg segment finite element model was created for Calabria segment 17 to assess the potential for damage due to the envelope of time-history seismic analysis force effects (tower legs were designed for the mean time-history force effects). Calabria segment 17 was found to have the maximum utilization ratio when assessed using the design verification methods. The modelled plate thicknesses and longitudinal stiffener dimensions are listed in Table 5-4. The segment is 20 m long and comprises six longitudinal panels of 3.333 m. The modelling procedures for applying the loads and considering the effects of initial imperfections and residual stresses are the same as those described in the preceding sections. The analysis is based on the smoothed multi-linear stress-strain relationship.

Plate	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**
Thickness	95	60	40	40	50	40	40	40
Longitudinal Stiffener	750 x 75	675 x 68	575 x 58	575 x 58	600 x 60	475 x 48	425 x 43	475 x 48

Table 5-4: Modelled plate thicknesses and longitudinal stiffener dimensions.

The critical seismic loads were caused by Sicilia time-history input 3, and occurred at time 15.59 s. The critical combined ULS loads at the base of the segment result from the min L001 linear combination and are:

Axial Load: Ns = 1730 MN

Longitudinal Moment: My = 7865 MNm

Transverse Moment: Mz = 160 MNm

Longitudinal Shear: Vy = 16 MN

Transverse Shear: Vz = 14 MN

The load-deformation response of the model (for axial shortening and longitudinal and transverse rotations) is shown in Figure 5-22, in which the load factor plotted on the vertical axis is the fraction of the specified loads that are applied in a particular load increment and the shortening and end rotation relative to theoretical elastic response at the specified loads is plotted on the horizontal axis. The black solid line at a load factor of 1.0 represents the load increment at which the specified loads are applied to the model. The diagonal black dashed lines represent increasing levels of flexibility from fully elastic (Secant 1X) to highly inelastic (Secant 3X).

The deviations of the load-deformation curves from the line representing the fully elastic response is due to the effects of residual stresses and geometric imperfections. At the specified loads, the axial shortening is approximately 20% larger and the rotations are approximately 30% larger than the theoretical elastic response. The analysis indicates that the ultimate capacity of the segment is approximately 14% higher than the specified loads. However, at this loading the secant stiffness has dropped so much that the actual moments are greater than the design values, so the effective increase in strength is less. The increased total moments were calculated by separating them into moments that are increased by reduced secant stiffness (moments from seismic action and buckling loads) and loads that are reduced by reduced secant stiffness (moments from restraint of the tower top by the cables). These moments were then factored to allow for the reduced secant modulus, which gave a modest increase in the total moment. The resistance of the section with these greater moments was calculated as described in the previous section for Sicilia tower leg segment 6.

At the maximum load sustained by the FE model, this strain was -4.298×10^{-3}. The strain arising from the increased moments (due to reduced secant stiffness) was calculated from the ratio of the increased moments to the moments applied to the FE model at each load factor. These increased strains are shown in the Figure 5-23. The maximum average strain reaches the limiting value at a load factor of approximately 1.11. This is a lower bound estimate of the actual section capacity, relative to the specified loads. The analysis indicates that the section can carry the maximum seismic demands that the design verifications had suggested were in excess of the section capacity.

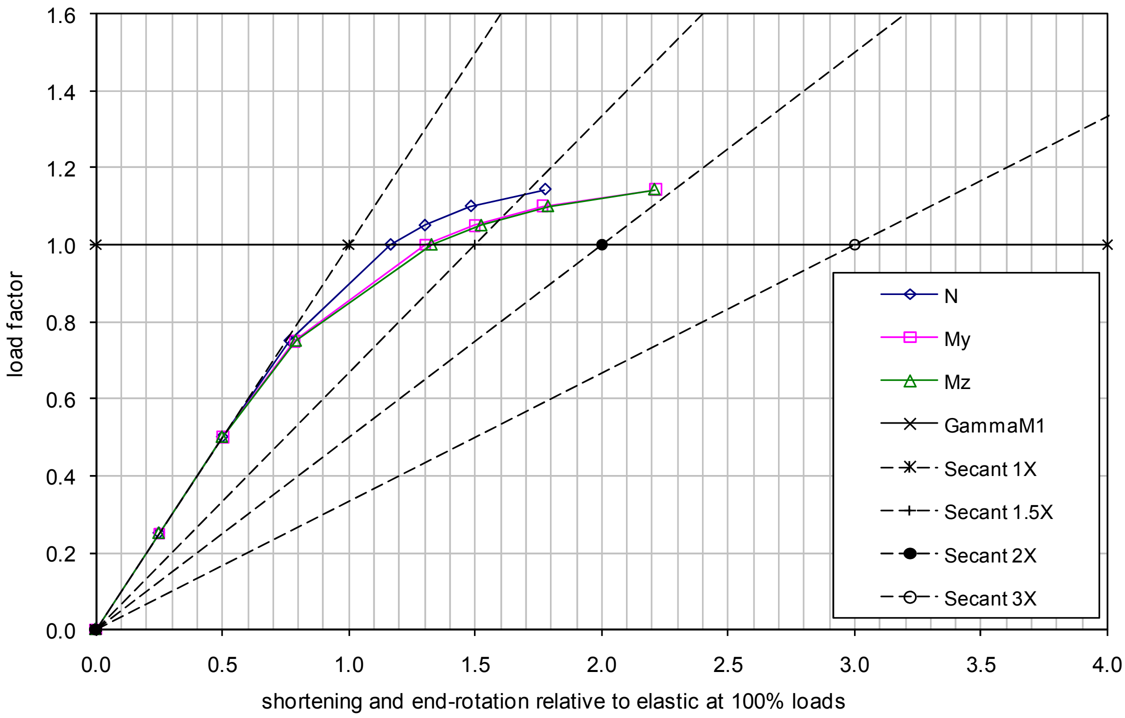


Figure 5-22: Load deformation response of Calabria tower leg segment 17 under the maximum seismic time-history load combination.

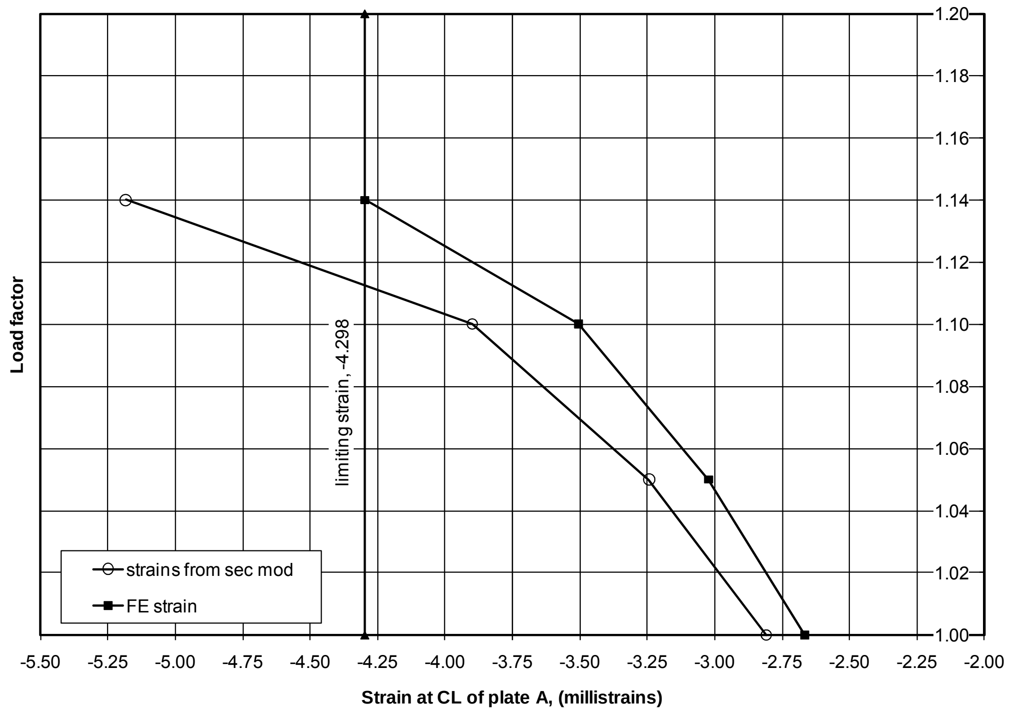


Figure 5-23: Calculated total strain compared with limiting strain (strain at failure).

The extent of damage that might be expected in the segment at the specified loads was evaluated considering the induced stresses and strains. Longitudinal stresses (axial) on the most compressed side of the segment under the specified loads are shown in Figure 5-24. All compressive stresses less than 350 MPa are plotted in the same colour so as to reduce the size and increase the number of stress intervals plotted close to the yield strength of the steel. The maximum reported compressive stress is 520 MPa and is shown to occur very locally at the intersection of plates A and B at the base of the segment on the right side of the figure. This stress is unreasonably high and not believed to be representative of the actual stress that would exist at this point, as the defined stress-strain curve is essentially perfectly plastic beyond the yield stress of 460 MPa and because the transverse moment causes higher compressive stresses at the opposite intersection of plates A and B. This higher than reasonable stress is likely a result of the boundary conditions at the connection of the rigid spider element through which load is applied to the model. Away from this point and at similar point on the other side of plate A the maximum compressive stresses are between 450 MPa and 475 MPa. Stresses of this magnitude occur in the lower panels on plates B and C and intermittently along the curved portion of plate B near its connection to plate A. Away from these areas, the maximum stresses are clearly below yield.

Because the steel was modelled as essentially perfectly plastic beyond the yield stress, stresses can not provide as effective an assessment of damage potential as can strains. Longitudinal strains (axial) on the most compressed face of the segment under the specified loads are shown in Figure 5-25. Strains less than 0.002 are plotted in the same colour so as to reduce the size and increase the number of strain intervals plotted close to the yield strain. The maximum reported compressive strain of 0.003827 occurs near the bottom of the section on the curved portion of plate B is and very localized. Away from this point, the maximum strains are generally between 0.003 and 0.00325. As indicated in Figure 5-3, these strains are all less than that corresponding to full yield (based on the 0.2% proof stress, which would give a full yield strain of 0.00419).

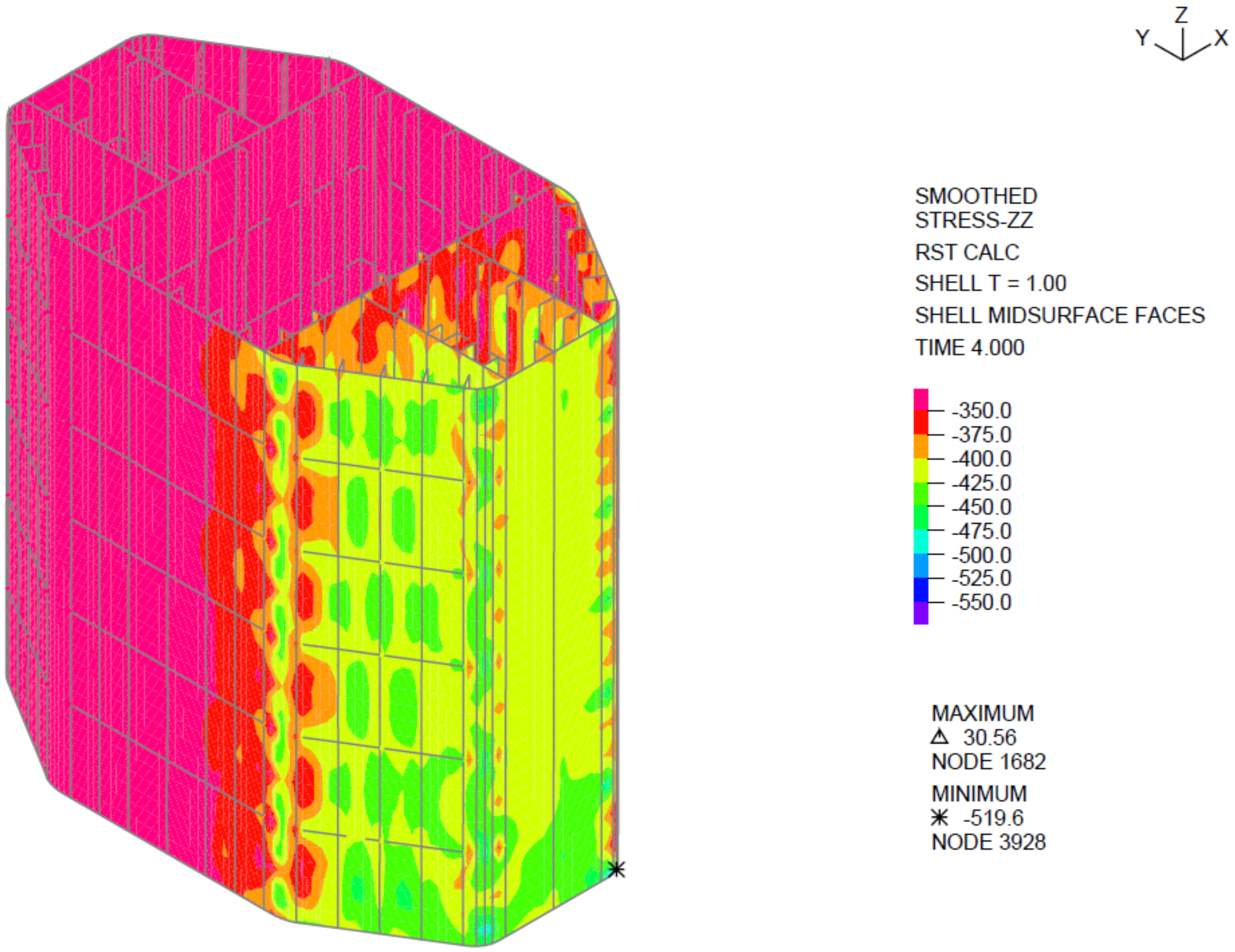


Figure 5-24: Calabria tower leg segment 17 longitudinal stresses due to the critical seismic load combination.

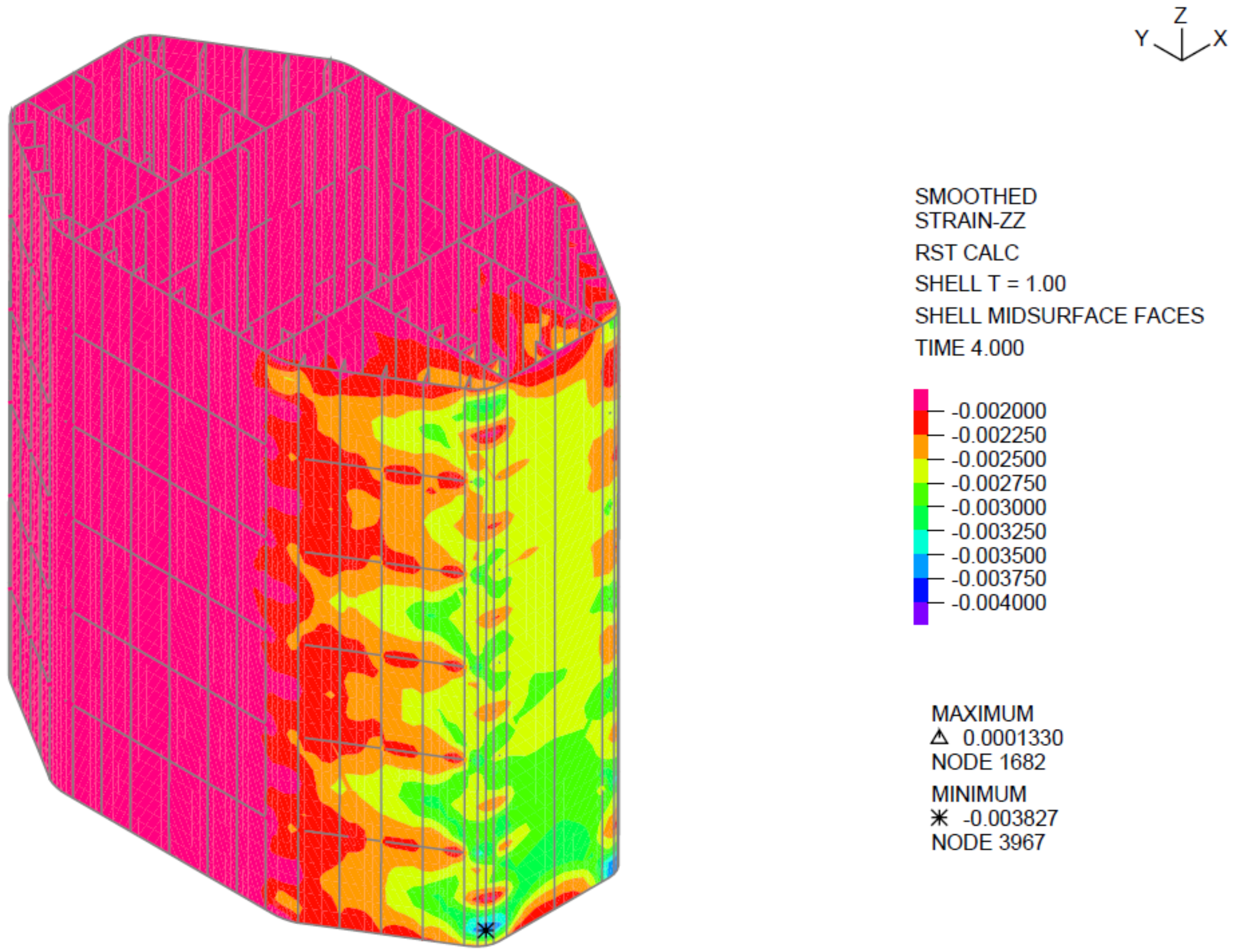


Figure 5-25: Calabria tower leg segment 17 longitudinal strains due to the critical seismic load combination.

Detailed views of the axial stresses and strains in the individual tower leg panels under the critical load combination are shown in Figure 5-26 to Figure 5-33 so that the effects of buckling of the longitudinal stiffeners on the outstand stresses can be more clearly seen. The panels shown are those in the most compressed tower leg quadrant (that shown in Figure 5-24 and Figure 5-25).

The stresses and strains in panel A, shown in Figure 5-26 and Figure 5-27, respectively, are relatively constant over the segment length with negligible variation through the longitudinal stiffener depth, indicating the absence of any local flexure that would be associated with a buckling-type behaviour. The stability of this panel as it approaches yield with a maximum compressive stress of 448 MPa is consistent with the design verifications for this element, which predict the full cross-section is effective up to yield.

The stresses and strains in panel B/C are shown in Figure 5-28 and Figure 5-29, respectively. There is a clear distinction in the behaviour of the plate B and C longitudinal stiffeners that is consistent with the differences in the plate thicknesses, stiffener depths and proximities to a stiff corner region. The plate B stiffener closest to the left side of the figure is well supported by the flexural stiffness of the corner region and has very little stress variation over the stiffener depth, indicating the lack of buckling-type behaviour. In contrast, the plate B stiffener closer to the middle of the panel and the more slender plate C stiffeners experience much more variation of stress through the stiffener depth indicating buckling-type behaviour between the transverse stiffeners. The compressive stress at the tip of the longitudinal stiffener outstand varies between adjacent panels, consistent with the sinusoidal shape the buckling stiffener. The maximum reported compressive stress of 488 MPa occurs at the bottom of the curved portion of plate B, but is believed to be related to the boundary conditions at the bottom of the segment, rather than being a realistic stress, as was noted in describing Figure 5-24. Away from this area the maximum compressive stresses are between 440 MPa and 460 MPa and are less than yield.

The stresses and strains in panel E/F are shown in Figure 5-30 and Figure 5-31, respectively. The behaviour of this panel is similar to that of panel B/C, with the indications of buckling-type behaviour of the longitudinal stiffeners increasing away from the stiff corner region at the intersection of plates A, B and E. Even though the average compressive stress on the 475 x 48 mm plate F stiffener is much less than those on the 600 x 60 mm plate E stiffener closest to plate A, buckling of the stiffener is much more prevalent because the stiffener is more slender. This panel is located away from the most compressed cross-section fibres and the maximum compressive stress is only 445 MPa and is less than yield.

The stresses and strains in panel G/H are shown in Figure 5-32 and Figure 5-33, respectively. Under the critical seismic loading the maximum compressive stress is 439 MPa and occurs locally at the tip of the plate G the longitudinal stiffener outstand in the third panel from the bottom. Both of plates G and H are only 40 mm thick with longitudinal stiffeners of 425 x 43 mm and 475 x 48 mm and will fail by buckling at an average compressive stress less than yield. The behaviour represented in the analysis results is consistent with the design verifications and the design philosophy of accepting reduced panel effectiveness away from the extreme fibres.

The detailed analysis of the tower leg segment shown by the design verifications to be the most heavily stressed under the maximum seismic force effects confirms the robustness of the design. All strains predicted by the analysis are less than that corresponding to full yield (as defined by the

0.2% proof stress) confirming that very limited inelasticity should be expected under the critical seismic load combination. Given that this is the most heavily stressed tower leg segment under the envelope of the time-history seismic analysis results, the behaviour of the adjacent tower leg segments for which design verifications also indicate utilization ratios slightly larger than 1.0, is expected to be similar but less severe. The analysis completed demonstrates that the tower legs meet the performance specification of Repairable Damage under ULS combinations, as specified in GCG.F.04.01 Table 6 and defined in Table 5 as “Occurrence of localised inelastic behaviour…”

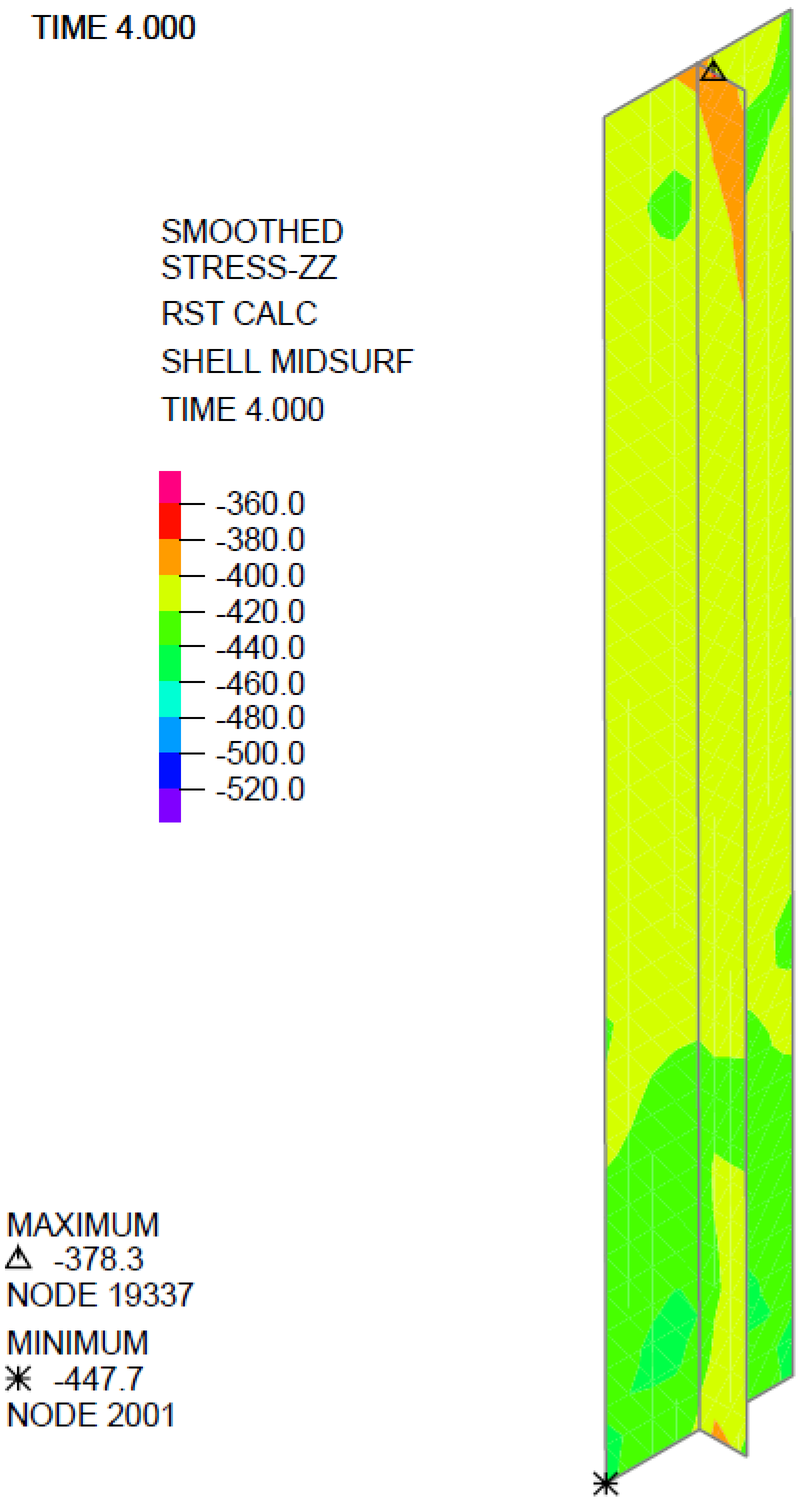


Figure 5-26: Calabria tower leg segment 17 plate A longitudinal stresses due to the critical seismic load combination.

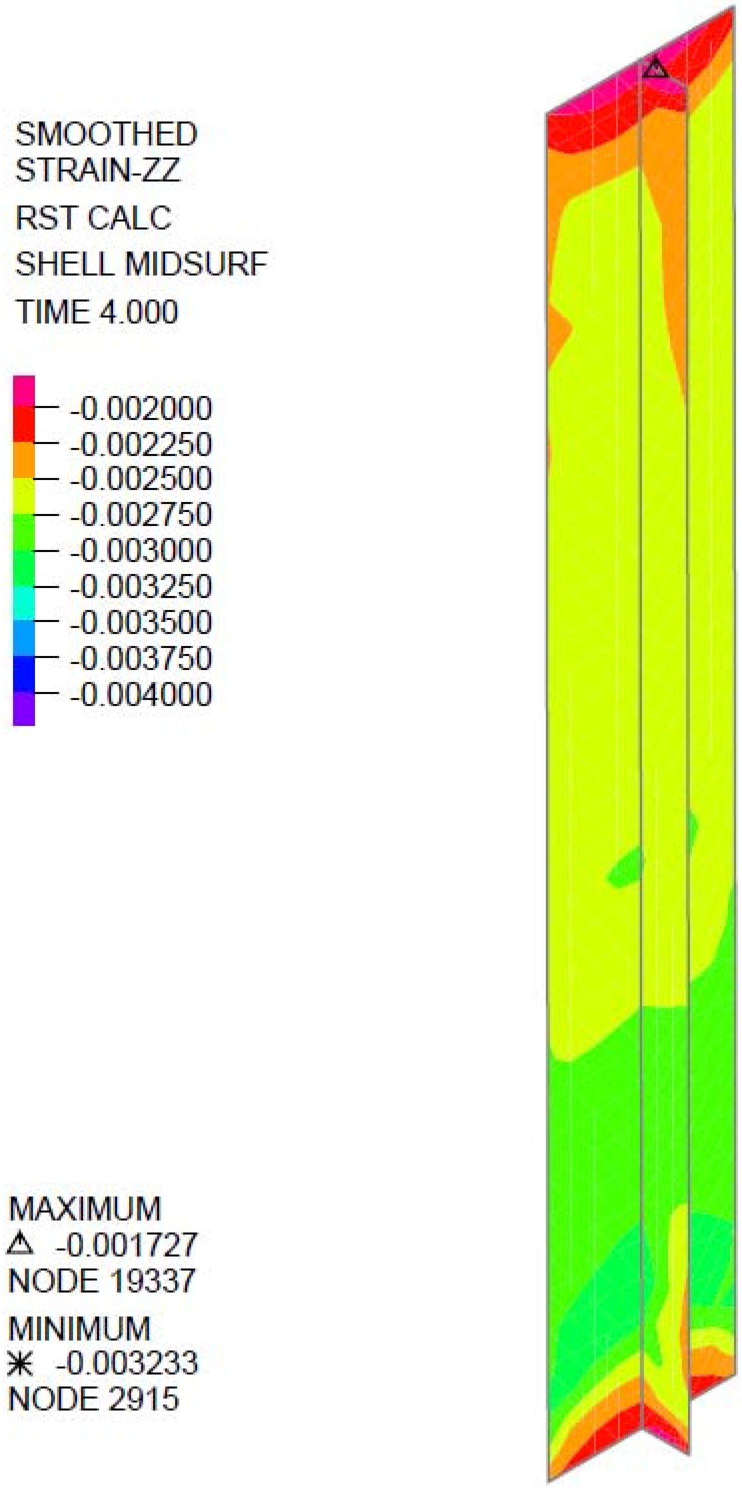


Figure 5-27: Calabria tower leg segment 17 plate A longitudinal strains due to the critical seismic load combination.

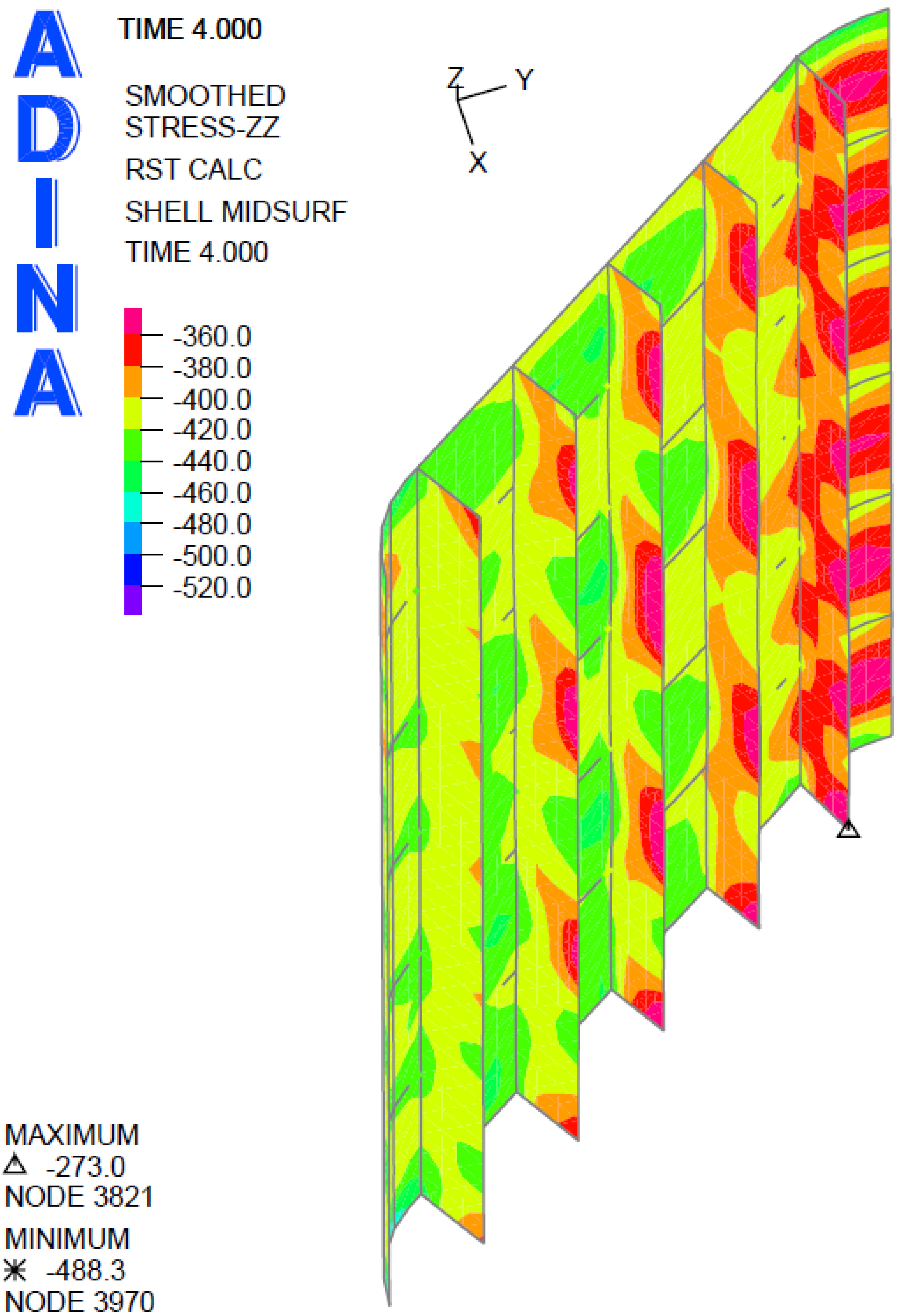


Figure 5-28: Calabria tower leg segment 17 plate B/C longitudinal stresses due to the critical seismic load combination.

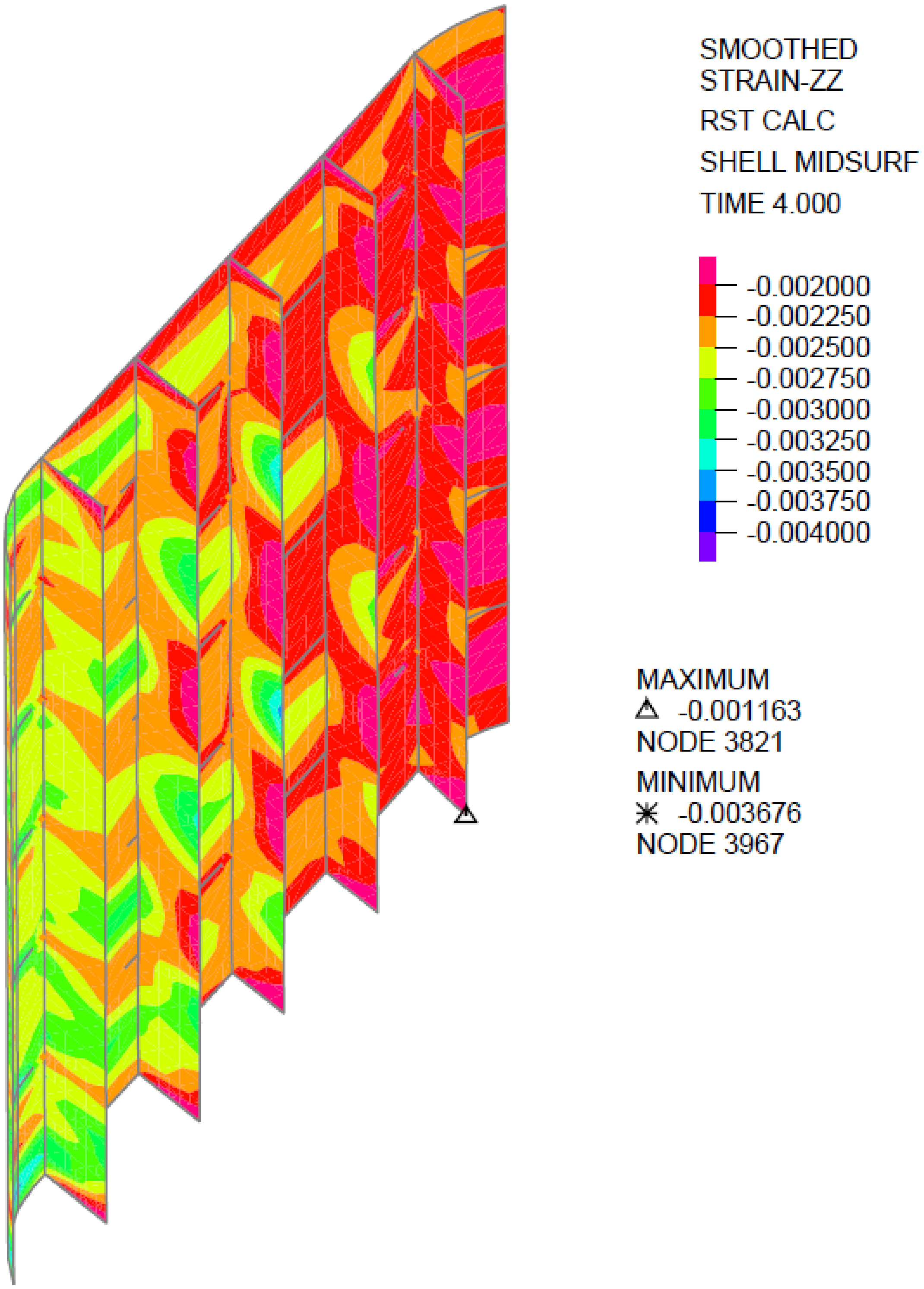


Figure 5-29: Calabria tower leg segment 17 plate B/C longitudinal strains due to the critical seismic load combination.

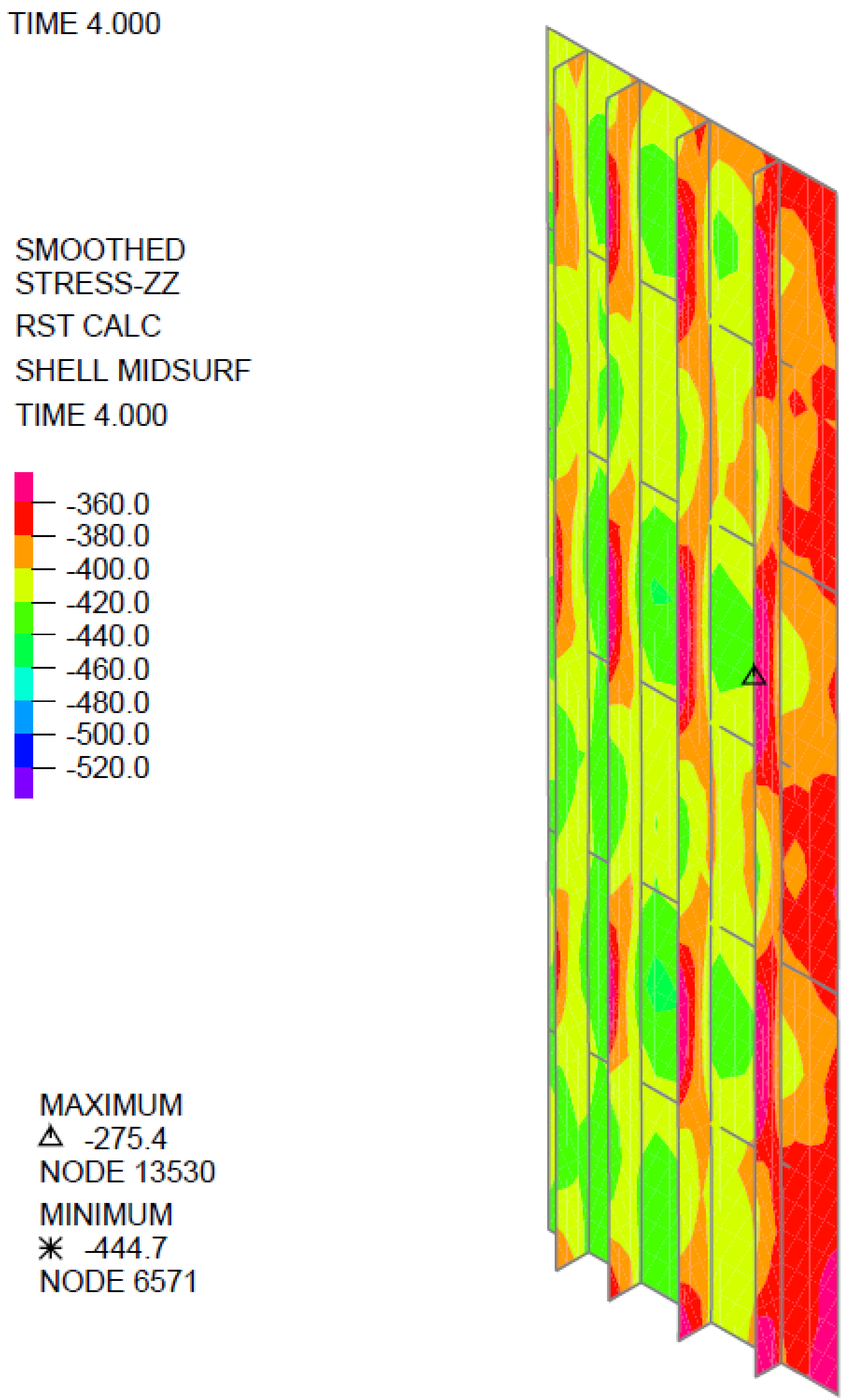


Figure 5-30: Calabria tower leg segment 17 plate E/F longitudinal stresses due to the critical seismic load combination.

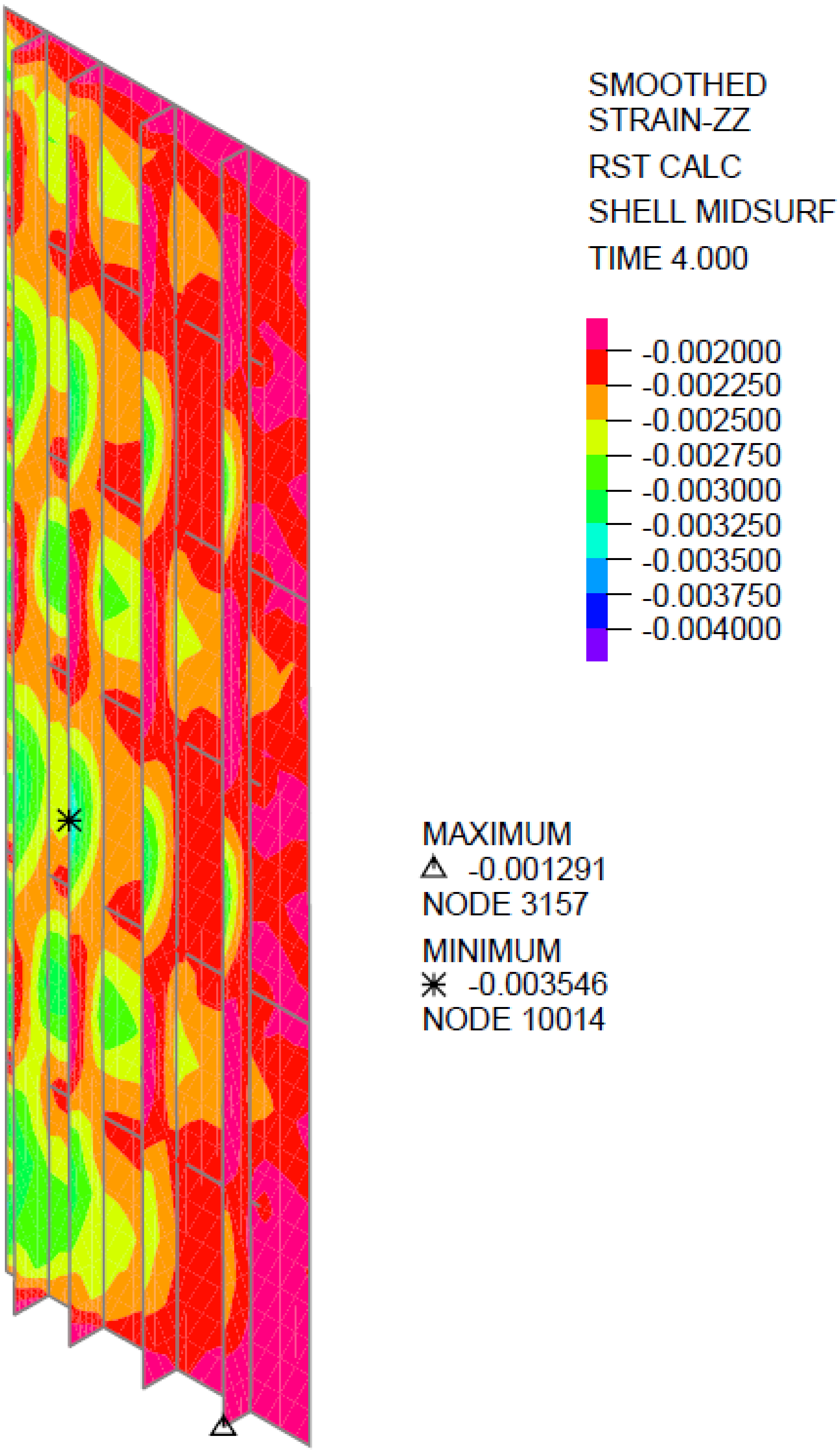


Figure 5-31: Calabria tower leg segment 17 plate E/F longitudinal strains due to the critical seismic load combination.

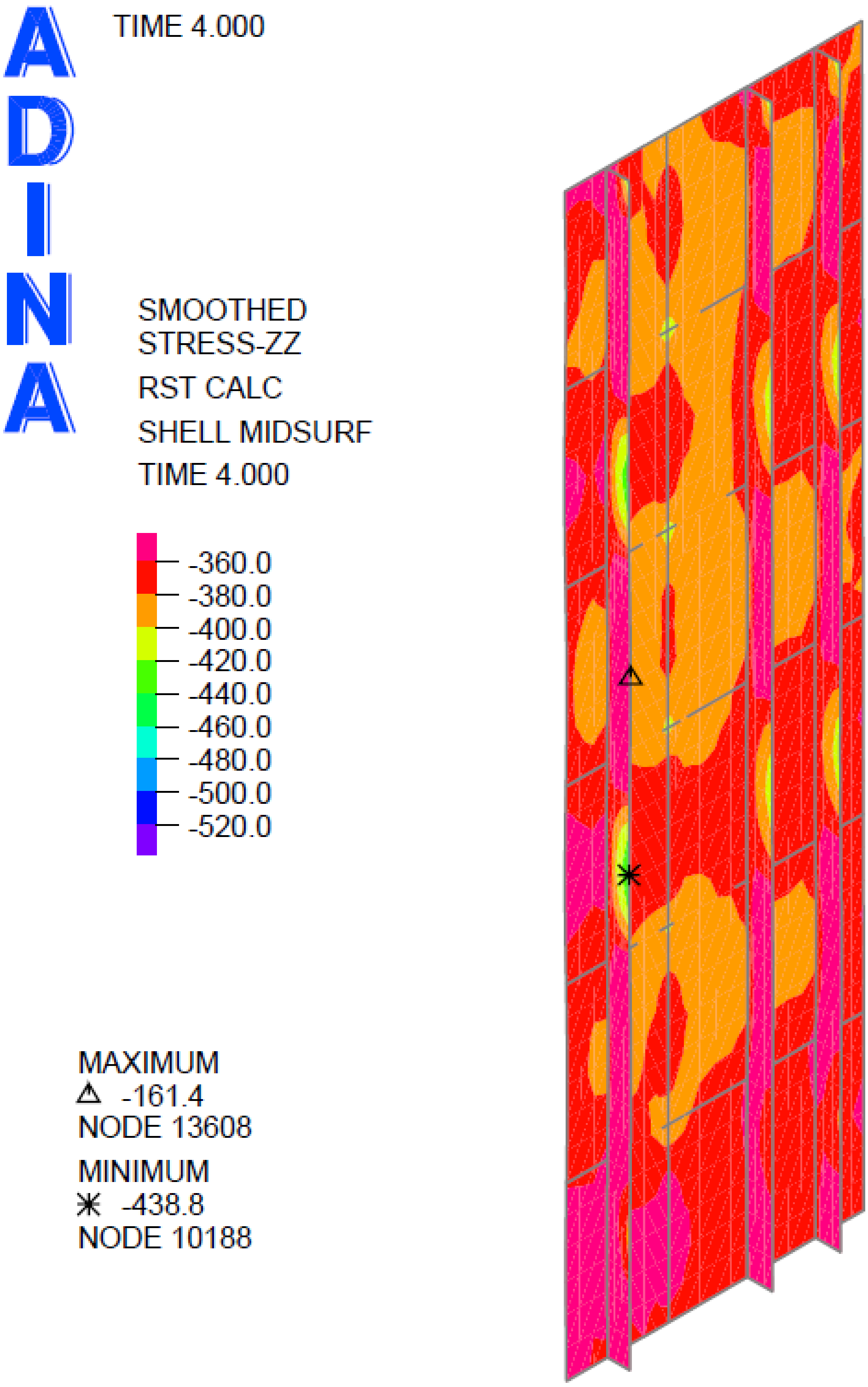


Figure 5-32: Calabria tower leg segment 17 plate G/H longitudinal stresses due to the critical seismic load combination.

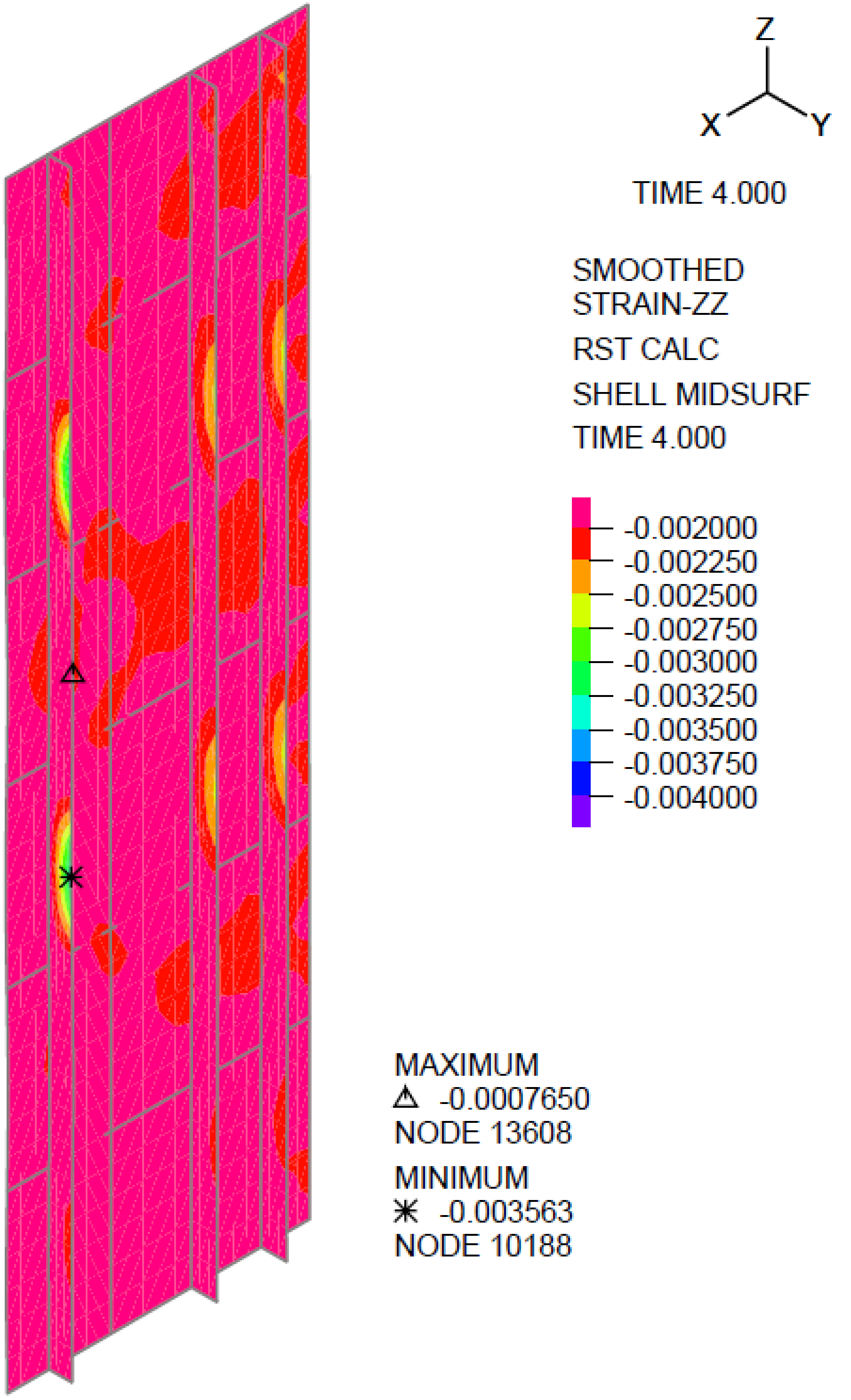


Figure 5-33: Calabria tower leg segment 17 plate G/H longitudinal strains due to the critical seismic load combination.

5.4 Fatigue

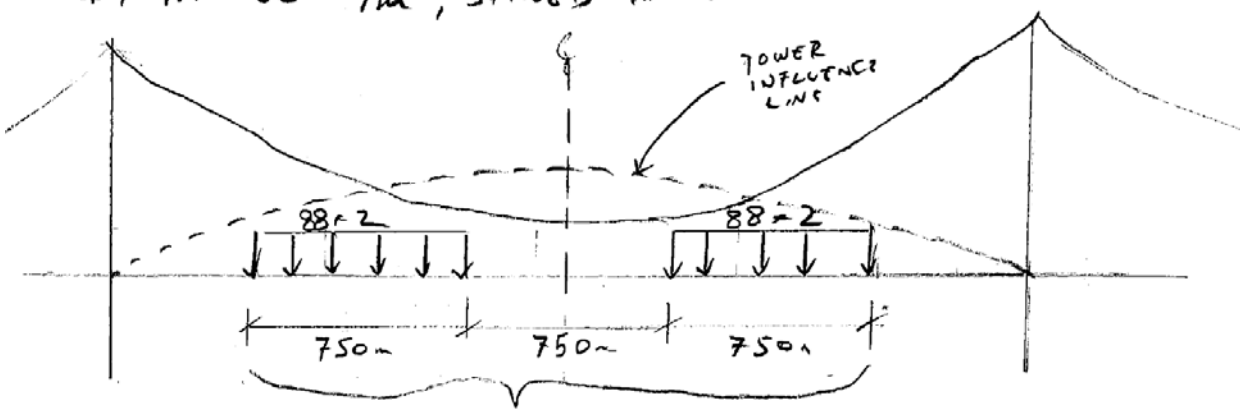
FATIGUE ASSESSMENT

THE TOWER WILL REMAIN FULLY IN COMPRESSION UNDER LIVE LOAD AND THE LIVE LOAD IS A VERY SMALL COMPONENT OF THE TOTAL STRESSES. THEREFORE, FATIGUE SHOULD NOT BE AN ISSUE. THE FOLLOWING IS SIMPLE HAND CALCS TO CONFIRM THIS ASSUMPTION

FROM IBDAS RESULTS, THE MAXIMUM STRESS RANGE DUE TO RAIL LOADING (LC 551) IS

σ_{RAIL} = 50 MPA ← Not checked

THIS STRESS RANGE IS DUE TO THE STANDARD "QA" LOADING WHICH CONSISTS OF UP TO 4 TRAINS, 750m LONG, AT 88 KN/m, SPACED 750 APART



PROBABLE TRAIN LOADING

WT OF 1 TRAIN = 88 KN/m × 750m = 66 MN

CONSERVATIVELY ASSUME THAT 1 TRAIN REPRESENTS 50% OF THE STRESS RANGE IF PLACED IN THE CENTER OF THE BRIDGE

∴ $\sigma_{1\ DESIGN\ TRAIN} = \frac{50\ MPa}{2} = 25\ MPa$

THE HEAVIEST FATIGUE TRAIN IS TYPE 5

$80 \text{ kN/m} \times 270\text{ m} = 21.6 \text{ MN}$

AT THE CENTER OF THE BRIDGE, THE INFLUENCE LINE IS VERY SMOOTH AND THE STRESSES IN THE TOWER WILL BE APPROXIMATELY PROPORTIONAL TO THE TOTAL WT OF THE TRAINS

$\therefore \sigma_{\text{1 TYPE 5 TRAIN}} = 25 \text{ MPa} \times \frac{21.6}{66 \text{ MN}} = 8.2 \text{ MPa}$

FOR FATIGUE STRENGTH THE CRITICAL CASE WILL HIGHER STRESS & FEWER CYCLES

CONSERVATIVELY ASSUME THAT ALL FATIGUE TRAINS ARE TYPE 5. (VERY CONSERVATIVE ASSUMPTION)
↳ MOST TRAINS ARE MUCH LIGHTER

ASSUME THAT 1/2 OF THE TRAINS OCCUR IN GROUPS OF 4 & THE OTHER 1/2 OCCUR IN GROUPS OF 2

$\sigma_{\text{4 FATIGUE TRAINS}} = 8.2 \text{ MPa} \times 4 = 32.8 \text{ MPa}$

$\sigma_{\text{2 FATIGUE TRAINS}} = 8.2 \text{ MPa} \times 2 = 16.4 \text{ MPa}$

TOTAL # OF TRAINS $67/\text{DAY} \times 2$ TRACKS $= 134/\text{DAY}$

$134/2 \cong 68/4 = 17$ 4 TRAIN GROUPS PER DAY

$66/2 = 33$ 2 TRAIN GROUPS PER DAY

TOTAL CYCLES

$17 \times 365 \times 200 = 1.24 \times 10^6$

$33 \times 365 \times 200 = 2.41 \times 10^6$

DETERMINE FATIGUE DAMAGE ASSUMING 36 MPa CATEGORY

4 TRAIN GROUPS

$\sigma = 32.8$ MPa

FATIGUE LIFE

$32.8^3 N_R = 36^3 \, 2 \times 10^6$

$N_R = 2.644 \times 10^6$ ✓

2 TRAIN GROUP

$\sigma = 16.5$ MPa

FATIGUE LIFE

$\sigma_D = .737 \sigma_c = 26.5$

$16.5^5 N_R = 26.5^5 \, 5 \times 10^6$

$N_R = 5.34 \times 10^7$ ✓

MINER'S SUMMATION

$$\frac{1.24 \times 10^6}{\left(\frac{2.644 \times 10^6}{1.35}\right)} + \frac{2.41 \times 10^6}{\left(\frac{5.34 \times 10^7}{1.35}\right)}$$

(1.35 = PARTIAL SAFETY FACTOR)

$0.633 + 0.061 = 0.7 < 1$

∴ FATIGUE WILL BE OK FOR A DETAILED ASSESSMENT USING ACTUAL TRAIN MIX ✓

5.5 Accidental Load Scenarios

5.5.1 Design Scenarios – Fire

INTRODUCTION

PURPOSE

The purpose of these calculations is to assess the impact of the Accidental Load Case of Fire

INPUT

Temperature input has been provided by COWI, Lyngby, as shown on pages 703-705.

Design is to be for Pool fires

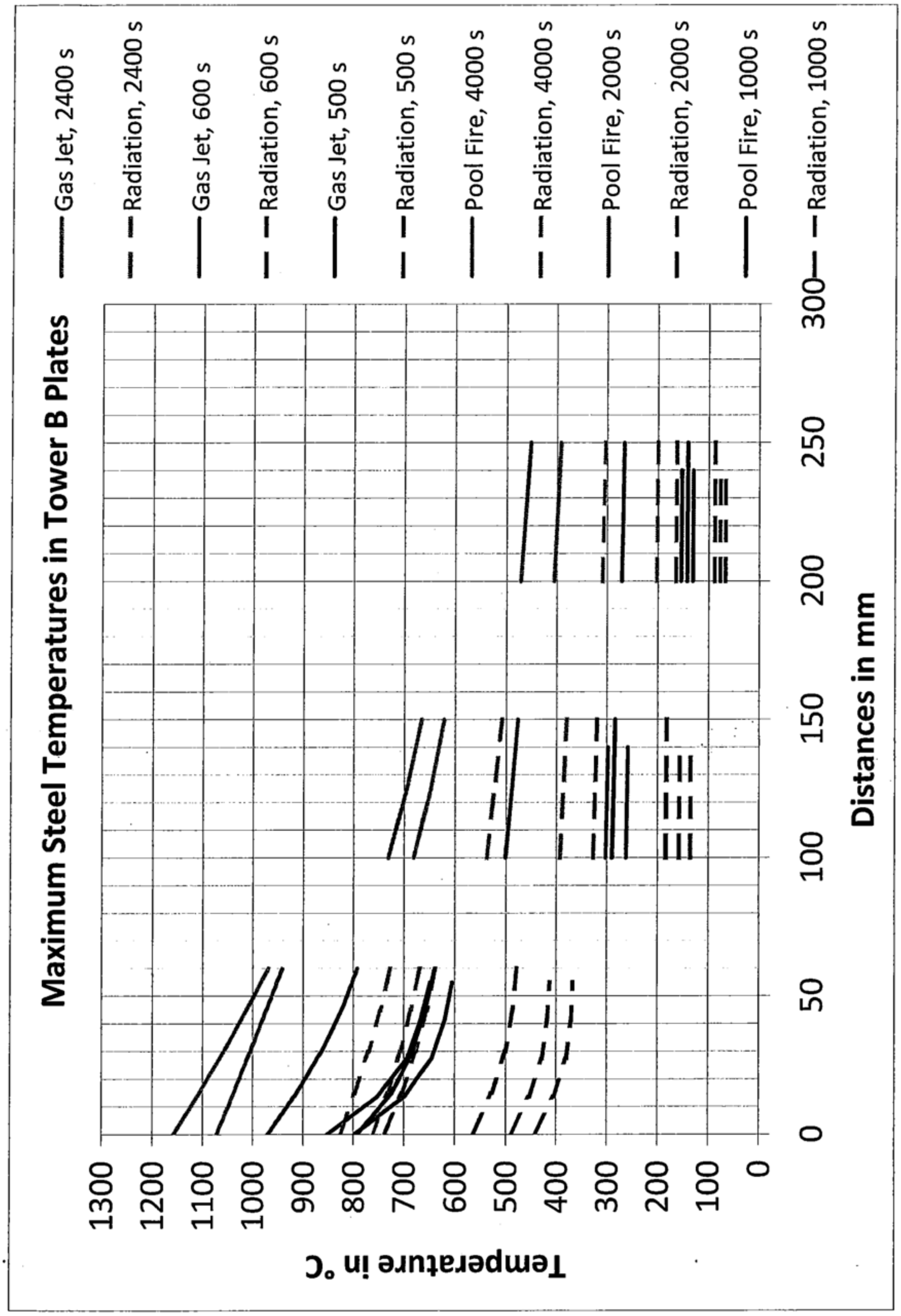
CODES

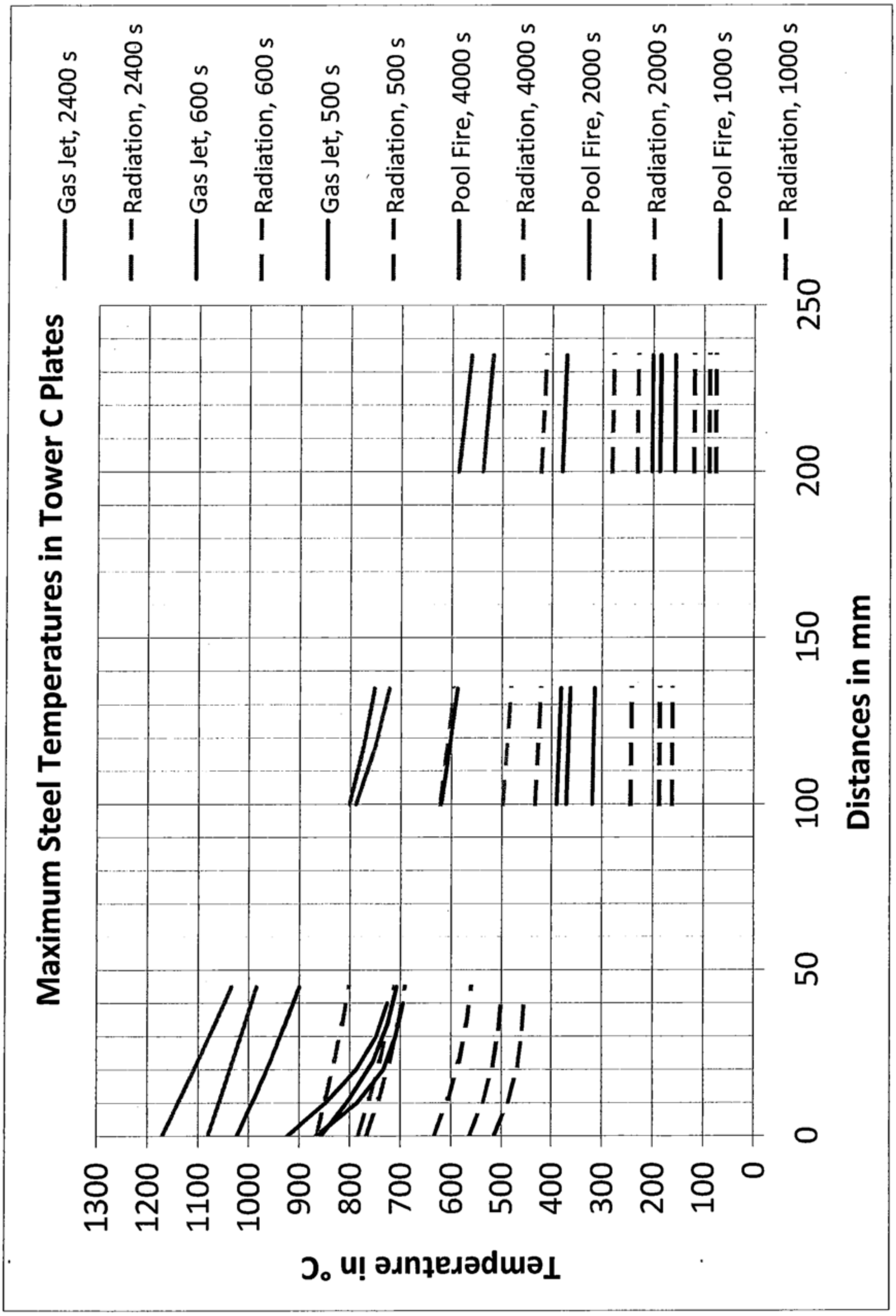
The effect will be studied using the Eurocode EN1993-1-2.
$\gamma_{M,fi}$ is taken @ 1.0 as recommended in #2.3 (NOTE)

CONCLUSIONS

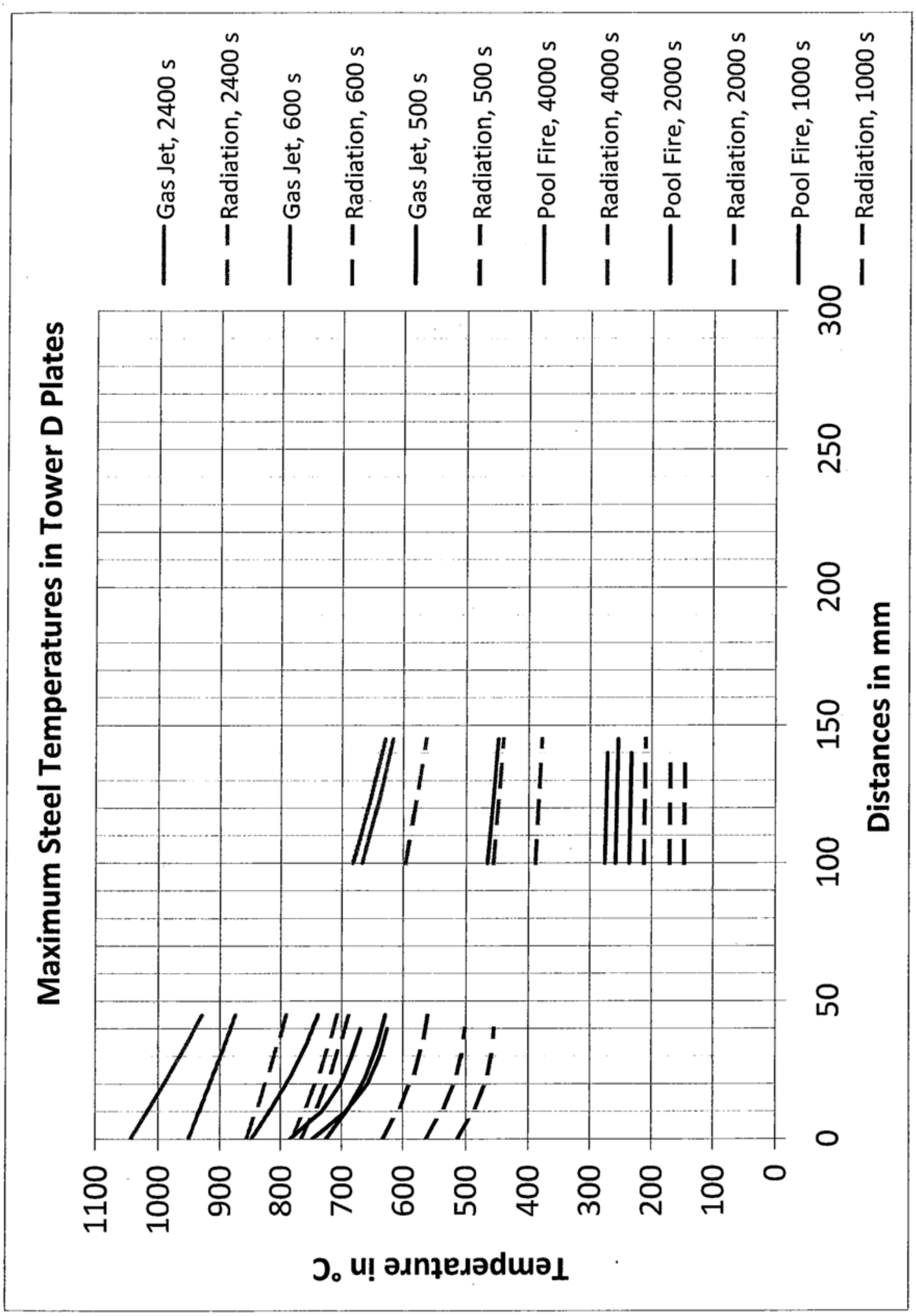
Even 300°C → big deflection (⇒ buckling) due to thermal expansion

(⇒ Need to protect steel to limit temperature rise)


Maximum Steel Temperatures in Tower B Plates
Gas Jet, 2400 s
Radiation, 2400 s
Gas Jet, 600 s
Radiation, 600 s
Gas Jet, 500 s
Radiation, 500 s
Pool Fire, 4000 s
Radiation, 4000 s
Pool Fire, 2000 s
Radiation, 2000 s
Pool Fire, 1000 s
Radiation, 1000 s
Temperature in °C
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
Distances in mm
0
50
100
150
200
250
300



Maximum Steel Temperatures in Tower C Plates
Gas Jet, 2400 s
Radiation, 2400 s
Gas Jet, 600 s
Radiation, 600 s
Gas Jet, 500 s
Radiation, 500 s
Pool Fire, 4000 s
Radiation, 4000 s
Pool Fire, 2000 s
Radiation, 2000 s
Pool Fire, 1000 s
Radiation, 1000 s
Temperature in °C
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
Distances in mm
0
50
100
150
200
250



Maximum Steel Temperatures in Tower D Plates
Gas Jet, 2400 s
Radiation, 2400 s
Gas Jet, 600 s
Radiation, 600 s
Gas Jet, 500 s
Radiation, 500 s
Pool Fire, 4000 s
Radiation, 4000 s
Pool Fire, 2000 s
Radiation, 2000 s
Pool Fire, 1000 s
Radiation, 1000 s
1100
1000
900
800
700
600
500
400
300
200
100
0
Temperature in °C
0
50
100
150
200
250
300
Distances in mm

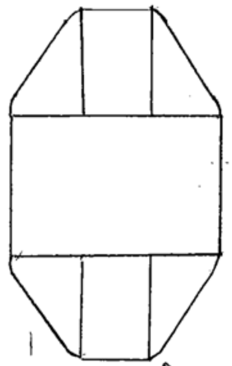
Out-of-Plane Deformations

pages 703 - 705

Pool fires → temperatures of 700°C – 1100°C

page 714

@300°C, $f_{y\theta} = 1.0 f_y$ & $E_\theta = 0.8 E_\theta$ ⇒ not too bad



BUT extension on 3.5m: $300 \times 12 \times 10^{-6} \times 3500 = 12.6$ mm

Assuming sinusoidal buckling, $\Delta l = \frac{\pi^2 \delta^2}{4l}$

$$\therefore \delta = \sqrt{\frac{4l \, \Delta l}{\pi^2}} = \sqrt{\frac{4 \times 3500 \times 12.6}{\pi^2}} = 134 \text{ mm}$$

⇒ enormous increase in imperfection ⇒ big drop of resistance.

5.5.2 Evaluation Scenarios

5.5.2.1 Impact

INTRODUCTION

PURPOSE

The purpose of this book is to assess the effect of aircraft impact on the tower
(i) locally
(ii) globally

INPUT

The loading specified is on page 803

Aircraft

We have verified, that there will not be requirements from the risk analysis. It is thus sufficient to fulfil the requirements from the current design basis, i.e. consider a 10 tonnes airplane at 600 km/h.

With the same methodology as used in the risk analysis for the Øresund bridge the calculation of the force/load time is as given below. The load area can be taken to be a circle with a diameter of 3 m (7m2), as for the Great Belt analysis.

Messina - aircraft collision			
Design basis			
	mass	10000	kg
	speed	600	km/h
	speed	167	m/s
Assumptions			
	length of airplane	10	m
Calculations			
	mass/length	1000	kg/m
	stopping time pr m	0.0060	sec per meter aircraft
Result			
	Force	27.8	MN
	total loading time	0.060	s
	Assumed load area	7	m2

Quasi-Static Force on Transverse Stiffener

Most sensitive panel is panel D because longest span and thinnest ⅊

Apply aircraft load to 1 transverse stiffener
As quasi-static load on transverse stiffener

Force = 27.8 MN on diameter of 3m at ℄

$V = \frac{27.8}{2} = 13.4$ MN

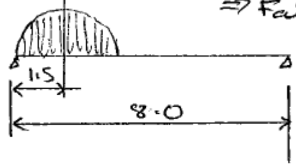
Shear resistance of web $= 18 \times 1100 \times \frac{460/\sqrt{3}}{1.05}$

$= 19800 \times 253 = 5.01$ MN ⇒ web shears $\left(\frac{D}{C} = \frac{13.4}{5} = 2.68\right)$
even with load at ℄

$\hat{M} \simeq 13.4 \times 3.8 = 50.9$ MN-m

$M_r \simeq 18 \times 280 \times 1100 \times 460 + 18 \times 340 \times 930 \times 460 = 2.55 \times 10^9 + 2.62 \times 10^9 = 5.17 \times 10^9 = 5.17$ MN-m

Force = 27.8 MN on diameter of 3m at 1.5m from end $\left(\Rightarrow D/C = \frac{50.9}{5.17} = 9.8\right)$ ⇒ Fail



$\text{Shear} = 27.8\left(1 - \frac{1.5}{8}\right) = 22.6$ MN

⇒ (Shear $\frac{D}{C} = \frac{22.6}{5.01} = 4.5$) ⇒ Complete failure

Moment $= 22.6 \times 1.5 = 33.9$ MN-m $\Rightarrow \frac{D}{C} = \frac{33.9}{5.17} = 6.6$

Shear resistance of 40⅊ @ 3m dia $= 40 \times \pi \times 3000 \times \frac{460/\sqrt{3}}{1.05}$

$= 377 \times 10^3 \times 253 = 95.4 \times 10^6$ N
$= 95$ MN $\Rightarrow \frac{D}{C} = \frac{27.8}{95} = 0.29$

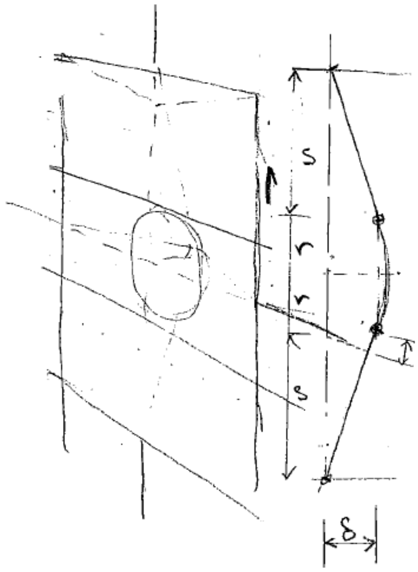
⇒ potentially the plate can resist the punching of the force, even though the transverse stiffeners will shear off.

Conclude that though plate can potentially resist punching, transverse stiffeners will fail by shear and bending, so resistance will depend on membrane action of plate + vertical stiffeners.

check that impulse behaviour is not much different

MEMBRANE STRESSES

Assume cross beams distribute load across panel and horizontal membrane stresses resist some load



∴ Consider as if whole impact resisted by 8.0m width

Change of length $= \frac{\delta^2}{s}$

Total length $= s + r$

$\therefore$ Strain $\varepsilon = \dfrac{\frac{\delta^2}{s}}{s+r} = \dfrac{\delta^2}{s(s+r)} \simeq \left(\dfrac{\delta}{s}\right)^2$

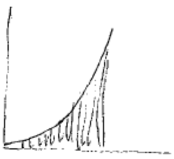
Resistance, $R = 2\frac{\delta}{s} \times \sigma A = 2\frac{\delta}{s} \times \varepsilon E A = 2\frac{\delta}{s} \times \frac{\delta^2}{s(s+r)} \times EA$

$EA = 210 \times 10^3 \times 8{,}000 \times 40 = 67.2 \times 10^9$

$210 \times 10^3 \times 5 \times 600 \times 60 = 37.8 \times 10^9$

Taking $s = 5000\,mm$

δ		R		ε
$\delta = 50\,mm$	→	$R = 2 \times 67.2 \times 10^9 \times \left(\frac{50}{5000}\right)^3$	$= 134.4\,N$;	0.1×10^{-3}
100	→	$R = 2 \times 67.2 \times 10^9 \times \left(\frac{100}{5000}\right)^3$	$= 1075\,kN$;	0.4×10^{-3}
200	→	$R = 2 \times 67.2 \times 10^9 \times \left(\frac{200}{5000}\right)^3$	$= 8601\,kN$;	1.6×10^{-3}
300	→	$R = 2 \times 67.2 \times 10^9 \times \frac{300}{5000}$	$= 29028$	3.6×10^{-3}
400	→		$68808\,kN$;	6.4×10^{-3}



Energy absorbed $= \int_0^\delta R\,dx = \int_0^\delta 2EA \frac{x^3}{s^3} dx = \frac{2EA}{s^3}\int_0^\delta x^3 dx$

$= \frac{2EA}{s^3}\frac{\delta^4}{4} = \frac{EA\delta^4}{2s^3}$

At $\delta = 200 \rightarrow U = \frac{EA(200)^4}{2s^3} = \frac{210 \times 10^3 \times 8000 \times 40\,(200)^4}{2(5000)^3}$ $\left(\sigma = 1.6 \times 10^{-3} \times 210 \times 10^3 = 336\,MPa\right)$

$= \frac{210 \times 10^3 \times 8 \times 10^3 \times 40 \times (0.2)^4 \times 10^{12}}{2 \times (5)^3 \times 10^9} = 0.43 \times 10^9\ N\text{-}mm = 0.43 \times 10^6\ N\text{-}m$

$\because kg\,m^2\,sec^{-2} = N\text{-}m$

Kinetic energy of aircraft $= \frac{1}{2}mv^2 = \frac{1}{2} \times 10 \times 10^3 \times (167)^2 = 0.139 \times 10^9\ N\text{-}m = 139 \times 10^6\ N\text{-}m$

$v = 600\,km/h = \frac{600 \times 10^3}{(60)^2} = \frac{600 \times 10^3}{3.6 \times 10^3} = 167\,m/sec$

$\sqrt[4]{\frac{0.139 \times 10^9}{0.43 \times 10^6}} = 4.24 \rightarrow \delta = 4.24 \times 200 = 848\,mm$

$\frac{210 \times 10^3 \times 8 \times 10^3 \times 40 \times (0.848)^4 \times 10^{12}}{2 \times (5)^3 \times 10^9} = 139 \times 10^9\ N\text{-}mm = 139 \times 10^6\ N\text{-}m$

$\varepsilon \simeq \frac{\delta^2}{s^2} = \left(\frac{848}{5000}\right)^2 = 0.0288$; 0.2% proof $= 0.002 \Rightarrow$ fails

MEMBRANE STRESSES (cont)

Strain $\varepsilon = \frac{\delta l}{l} = \frac{\frac{\delta^2}{s}}{(s+r)} = \frac{\delta^2}{s(s+r)} \simeq \left(\frac{\delta}{s}\right)^2$

Resistance to horizontal force $R = 2 \times \frac{\delta}{s} \sigma A$

$A = 8000 \times 40 = 320000 \text{ mm}^2$

$+ 5 \times 600 \times 60 = 180000$

$500{,}000 \text{ mm}^2$

$= 2 \times \left(\frac{\delta}{s}\right) \varepsilon E A$

$= 2 \times \left(\frac{\delta}{s}\right)\left(\frac{\delta}{s}\right)^2 E A = 2\left(\frac{\delta}{s}\right)^3 E A$

$\therefore$ Energy absorbed $= \int_0^\delta R\, d\delta = \frac{2EA}{s^3}\int \delta^3 d\delta = \frac{2EA}{s^3}\frac{\delta^4}{4} = \frac{EA\delta^4}{2s^3}$

$s = 20 \times 10^3 \text{ mm} \rightarrow \frac{EA}{2s^3} = \frac{210 \times 10^3 \times 500 \times 10^3}{2 \times (20)^3 \times 10^9} = 6.56 \times 10^{-3}$

			E
$\delta = 400 \rightarrow \frac{EA\delta^4}{2s^3}$	$= 6.56 \times 10^{-3} \times (400)^4 = 0.168 \times 10^9$ N-mm	$= 0.168 \times 10^6$ N-m	
$\delta = 600$	$6.56 \times 10^{-3} \times (600)^4 = 0.850 \times 10^9$	$= 0.85 \times 10^6$	
$\delta = 800$	$(\quad) \times (800)^4 = 2.69 \times 10^9$	$= 2.69 \times 10^6$	
$\delta = 1000$	$(\quad) \times (1000)^4 = 6.56 \times 10^9$	$= 6.56 \times 10^6$	2.5×10^{-3}
$\delta = 1200$	$(\quad) \times (1200)^4 = 13.6 \times 10^9$	$= 13.6 \times 10^6$	3.6×10^{-3}
$\delta = 1400$	$(\quad) \times (1400)^4 = 25.2 \times 10^9$	$= 25.2 \times 10^6$	4.9×10^{-3}
$\delta = 1600$	$(\quad) \times (1600)^4 = 43.0 \times 10^9$	$= 43.0 \times 10^6$	
$\delta = 1800$	$(\quad) \times (1800)^4 = 68.9 \times 10^9$	$= 68.9 \times 10^6$	
$\delta = 2000$	$(\quad) \times (2000)^4 = 105 \times 10^9$	$= 105 \times 10^6$	
$\delta = 2145$	$(\quad) \times (2145)^4 = 139 \times 10^9$	$= 139 \times 10^6$	0.0115
$\delta = 2200$	$(\quad) \times (2200)^4 = 154 \times 10^9$	$= 154 \times 10^6$	

Strain @ 2145 $= \left(\frac{\delta}{s}\right)^2 = \left(\frac{2145}{20000}\right)^2 = 0.0115 = 1.15\%$ strain $< 5\%$ max forming pressure vessels

$\sqrt{\frac{0.002}{0.0115}} = 0.417 \therefore 0.417 \times 2145 = 895$

Strain @ 0.2% proof $= \frac{460}{210 \times 10^3} + 2 \times 10^{-3} = 2.2 + 2 = 4.2 \times 10^{-3}$
$= 0.0042$

Trial length of 20m was chosen because page 811 showed $D/c = 9.8$ for stiffeners with 280 x 18. flanges. These flanges will be increased for stability, so the number of cross beams to resist the impact might reduce to about 6 N°, which at 3.33 centres minimum $\Rightarrow$ 20 metres O/A height.

2x20 m = 40m overall height appears to be minimum

MEMBRANE STRESSES (cont)

Assuming 20 m high damage zone

If criterion is resistance & quasi-static force,

At $\delta = 1m$; Reaction $R = 2\left(\frac{\delta}{S}\right)^3 EA = 2\left(\frac{1}{20}\right)^3 210\times10^3 \times 500\times10^3 = 26.3\ MN \rightarrow \frac{D}{C} = \frac{27.8}{26.3} = 1.05$

(← page 816)

Force = rate of change of momentum

Momentum = mv = 1000 × 167 = 167000 kg m/sec per metre of plan

∴ Rate of change of momentum $= \frac{\text{Momentum/m}}{\text{Stopping time/m}} = \frac{167000}{0.0060} = 27.8\times10^6$

$\frac{\text{kg m/sec}}{\text{sec}} = \text{kg m/sec}^2 = N$

∴ Force = 27.8 MN <u>constant</u> force by these assumptions

MEMBRANE STRESSES (cont)

Try 80m length of membrane action $\Rightarrow s = \frac{80}{2} = 40$ metres

So Energy absorbed (as page 816) $= \frac{EA\delta^4}{2s^3}$ — see page 816

$\therefore$ @ s=40m, $\frac{EA}{2s^3} = \frac{210\times10^3 \times 500\times10^3}{2\times(40)^3\times10^9} = 0.82\times10^{-3}$

@ $\delta = 4.0m = 4000\,mm$ energy $= 0.82\times10^{-3}\times(4000)^4 = 210\times10^9\,N\text{-}mm = 210\times10^6\,N\text{-}m$

$\therefore$ Required elastic deflection $= \sqrt[4]{\frac{139\times10^6}{210\times10^6}}\times4000 = 3.61$ metres, ← Kinetic energy of aircraft on page 815

Strain $= \left(\frac{\delta}{s}\right)^2 = \left(\frac{3610}{40000}\right)^2 = 8.15\times10^{-3} \gg 4.2\times10^{-3}$ at 0.2% proof stress

$\Rightarrow$ Membrane force will certainly reach yeild force of skin + stiffeners, and the yeild force will be increased by strain rate and might be higher than expected because steel yeild usually is at least 15% greater than specified minimum.

$\therefore$ 80 m length of membrane seems possible

$\therefore$ Assume applied force $= 1.5\times$ nominal yeild force

$= 1.5\times500{,}000\times460 =$ 345 MN

Find Deflection @ $1.5\times$ nominal yeild

$\therefore$ Strain $= \frac{\sigma}{E} + 0.2\%$ proof $= \frac{1.5\times460}{210000} + 2\times10^{-3} = 3.29\times10^{-3} + 2\times10^{-3} = 5.29\times10^{-3}$

Strain $= \left(\frac{\delta}{s}\right)^2 \rightarrow \frac{\delta}{s} = \sqrt{\text{Strain}}$ (page 816)

$\rightarrow \delta = s\times\sqrt{\text{Strain}} = 40\times\sqrt{5.29\times10^{-3}} =$ 2.91 metres

$\therefore$ Energy absorbed by elastic behaviour to 2.91 metres $= \frac{EA\delta^4}{2s^3}$

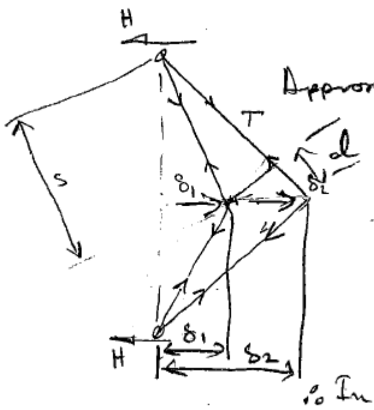
$= \left(\frac{EA}{2s^3}\right)\delta^4 = 0.82\times10^{-3}(2.91)^4\times10^{12} = 58.8\times10^9\,N\text{-}mm = 58.8\times10^6\,N\text{-}m$

$\therefore$ Energy to be absorbed by plastic deformation $= 139\times10^6\,N\text{-}m$ (page 815)
$- 59\times10^6$
$= 80\times10^6\,N\text{-}m$

$\therefore$

MEMBRANE STRESSES (cont)

PLASTICITY



Energy absorbed = Force × distance moved
Ignore change of length S for geometry
Approx: Distance moved, d = $\frac{(\delta_2-\delta_1)}{S} \times (\delta_2-\delta_1)$

∴ Energy = 2 sides × yield force T × $\frac{(\delta_2-\delta_1)^2}{S}$

(page 818)

= 2 × 345 MN × $\frac{(\delta_2-\delta_1)^2}{40m}$

∴ In N & metres = $\frac{2 \times 345\times10^6}{40} \times (\delta_2-\delta_1)^2 = 17.25\times10^6 (\delta_2-\delta_1)^2$

(page 818)

Energy to be absorbed beyond elastic limit = 80×10^6 N-m

∴ $17.25\times10^6 (\delta_2-\delta_1)^2 = 80\times10^6$

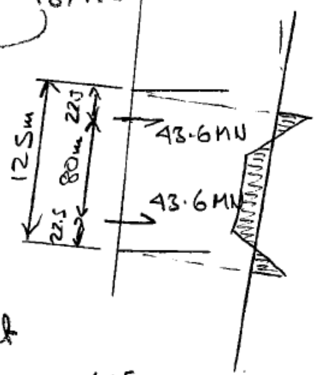
∴ $(\delta_2-\delta_1) = \sqrt{\frac{80\times10^6}{17.25\times10^6}} = 2.15m$

From page 818, elastic deflection δ_1 = 2.91 m

∴ Maximum deflection δ_2 = 5.06 m

∴ Horizontal component H = $\frac{\delta_2}{S} \times T = \frac{5.06}{40} \times 345 = 43.6$ MN

∴ Total moment (max sag & max hog) = 22.5 × 43.6 = 981 MN-m



$\frac{Wab^2}{L^2} = \frac{W\ 22.5\times(102.5)^2}{125^2} = 15.13W$

$\frac{Wa^2b}{L^2} = \frac{W(22.5)^2\times(102.5)}{125^2} = 3.32W$

18.45W = Fixed Ended Moment

Wa = 22.5W = Simply supported moment

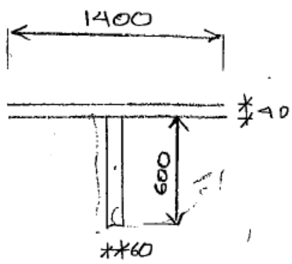
∴ SS − FEM = 22.5W − 18.45W = 4.05W ⇒ $\frac{4.05}{22.5}$ = 0.18 × SS

Some flexibility in cross beams, but little ⇒ assume Min span = 0.25 × SS
= 0.25 × 981 = 245 MN-m

ENERGY IN PLASTIC ROTATIONS

Vertical stiffeners

Find plastic modulus W_{pl}



Area of skin $= 1400 \times 40 = 56000\ mm^2$

stiffener $= 600 \times 60 = 36000$

$92\,000\ mm^2$

$\frac{A}{2} = \frac{92000}{2} = 46000\ mm^2$

$\therefore$ Plastic neutral axis in plate at $\frac{46000}{1400} = 32.9\ mm$

$\therefore W_{pl} \doteq 600 \times 60 \left(300 + 32.9\right) = 11.94 \times 10^6\ mm^3$

$1400 \times 32.9 \times \frac{32.9}{2} = \underline{0.76}$

12.70

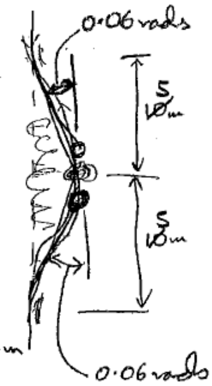
$\therefore M_p/\text{stiff} = W_{pl} \frac{f_y}{\gamma_M} = 11.94 \times 10^6 \times 460 = 5.49 \times 10^9\ N\text{-}mm = 5.49 \times 10^6\ N\text{-}m$

use $\gamma_M = 1.0$ for accident

Kinetic energy per 1.4m width × 10m height $1.4 \times 10 \times 15.2 \times 10^6 = 213 \times 10^6\ N\text{-}m$

$\therefore$ Rotation required to absorb energy $= \frac{213 \times 10^6}{5.49 \times 10^9} = \frac{213}{5490} = 0.039$ rads

$= 0.039 \times \frac{180}{\pi} = 2.23°$



Assuming 20m damage height

$0.039 \times 10m = 0.39$ metres

At 300 mm displacement in 5m $\rightarrow \theta = \frac{300}{5000} = 0.06$ radians

$\therefore$ Plastic rotation energy $= 5.49 \times 10^6 \times 0.06 = 0.33 \times 10^6\ N\text{-}m$

For 5 stiffeners with 2 hinges $\rightarrow 2 \times 5 \times 0.33 \times 10^6 = 3.3 \times 10^6\ N\text{-}m$

Aircraft impact energy $= 139 \times 10^6\ N\text{-}m \leftarrow$ see page 815 $\rightarrow \frac{D}{C} = \frac{139}{3.3} = 42$!!

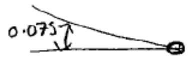
Transverse stiffener

W_{pl} of transverse stiffener $\doteq 280 \times 18 \times 1100 = 5.54 \times 10^6\ mm^3$

$+ 340 \times 18 \times 930 = \underline{5.69 \times 10^6}$

$11.23 \times 10^6\ mm^3$

$M_p = 11.23 \times 10^6 \times 460 = 5.17 \times 10^9\ N\text{-}mm = 5.17 \times 10^6\ N\text{-}m$



At 300 mm displacement in 4m $\rightarrow \theta = \frac{300}{4000} = 0.075$ radians

$\therefore$ Plastic rotation for both sides of ℄ $= 2 \times 5.17 \times 10^6 \times 0.075 = 0.78 \times 10^6\ N\text{-}m$

RESISTANCE OF DAMAGED TOWER

Assume impact in segment 11

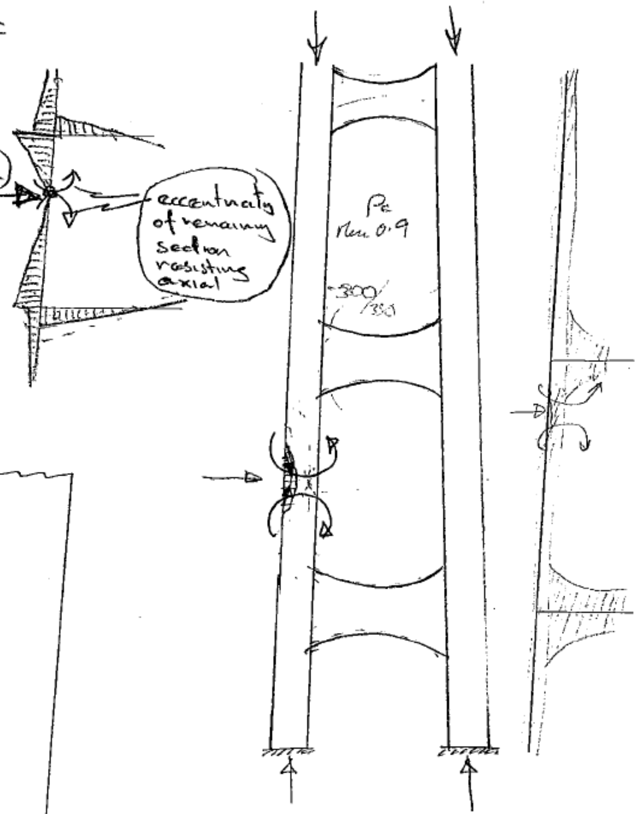
BENDING MOMENTS

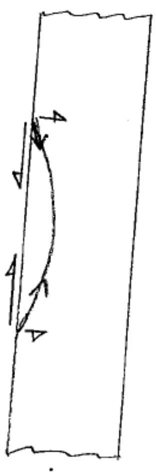
Impact Bending Moments

2 ways to look at impact BMD

1) Classic bending moments for member of reduced capacity at damage

2) Torsion from membrane action





Other Bending Moments

1) Buckling moments from coincident vertical loads

Axial

Factored dead + 1.0 × QL   GCG.F.04.01   #5.2   ½ Rail ⅓ Road

Unfactored axial mid-height between Cross Beams 1 & 2 = 1200 MN/leg

Factor by 1.15 & 1.25 — say 1.20

1.20 × 1200 = 1440 MN

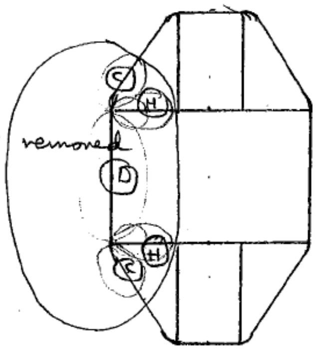
Resistance say 0.95 yield.

RESISTANCE OF DAMAGED TOWER (cont)

IMPACT ON PLATE D

Assume damage removes the resistance of

1) plate D
2) plates H on the impact side of the leg
3) plates C "



Calculate elastic section properties

From page 823, shift of neutral axis = 1.335 metres

area remaining = 5.27×10^6 mm²

∴ Axial resistance @ $p_c = 0.95$ & $\gamma_m = 1.0$ = $0.95 \times \frac{460}{1.0} \times 5.27 \times 10^6 = 2300$ MN

Axial load = 1440 MN factored dead load

\+ 345 MN (150% yield force of plate D + stiffs)

1785 MN

∴ Utilisation resisting axial alone = $\frac{1785}{2300} = 0.776$ → 0.776

∴ Available for bending = $(1 - 0.776) = 0.224$

∴ Moment of resistance = $\hat{\sigma}\frac{I}{y} = \left(0.224 \times 0.95 \times \frac{460}{1.0}\right) \times \frac{45.2 \times 10^{12}}{6000}$ (page 823)

$= 97.9 \times \frac{45.2 \times 10^{12}}{6 \times 10^3} = 737 \times 10^9$ N-mm = 738 MN-m

Moment calculated on page 818A = 245 MN-m in the damaged length of the leg

⇒ Utilisation for bending from impact ≤ $\frac{245}{738} \times 0.224$ = → 0.074

sub Buckling moment ≃ 1440 × 1.059m = 1525 MN-m

at damage ∴ Stress = $\frac{My}{I} = \frac{M \times 7000}{300 \times 10^{12}} = \frac{1525 \times 10^9 \times 7 \times 10^3}{300 \times 10^{12}}$ = 35.6 MPa → Utilisation = $\frac{35.6}{0.95 \times 460}$ = 0.081

0.931

≯ 1.0

NOTE Higher moment in undamaged portion is largely within the depth of the cross-beam connection ⇒ OK. ⇒ OK (just!)

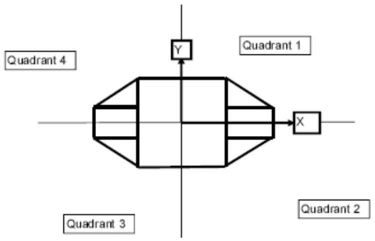


plate	quadrant			tp	bp	ns	ts	bs	Api	Asi	Xopi	Yopi	Xosi	Yosi	Api*Xopi	Api*Yopi	Asi*Xosi	Asi*Yosi	XNApi	YNApi	XNAsi	YNAsi	Api*XNApi^2	Api*YNApi^2	Asi*XNAsi^2	Asi*YNAsi^2
A	1&2		tA	75	4,500	1	75	750	337,500	56,250	9962.5	0	9550	0	3362343750	0	537187500	0	9962.5	0	9550	0	3.34973E+13	0	5.13014E+12	0
A	3&4			75	4,500	1	75	750	337,500	56,250	-9962.5	0	-9550	0	-3362343750	0	-537187500	0	-9962.5	0	-9550	0	3.34973E+13	0	5.13014E+12	0
B		1	tB	65	3,432	2	68	675	223,105	91,800	9062.5	3187.5	9062.5	3187.5	2021887848	711146760.5	831937500	292612500	9062.5	3187.5	9062.5	3187.5	1.83234E+13	2.26678E+12	7.53943E+12	9.32702E+11
B		2		65	3,432	2	68	675	223,105	91,800	9062.5	-3187.5	9062.5	-3187.5	2021887848	-711146760.5	831937500	-292612500	9062.5	-3187.5	9062.5	-3187.5	1.83234E+13	2.26678E+12	7.53943E+12	9.32702E+11
B		3		65	3,432	2	68	675	223,105	91,800	-9062.5	-3187.5	-9062.5	-3187.5	-2021887848	-711146760.5	-831937500	-292612500	-9062.5	-3187.5	-9062.5	-3187.5	1.83234E+13	2.26678E+12	7.53943E+12	9.32702E+11
B		4		65	3,432	2	68	675	223,105	91,800	-9062.5	3187.5	-9062.5	3187.5	-2021887848	711146760.5	-831937500	292612500	-9062.5	3187.5	-9062.5	3187.5	1.83234E+13	2.26678E+12	7.53943E+12	9.32702E+11
C		1	tC	40	3,432	3	60	600	137,295	108,000	7187.5	5062.5	7187.5	5062.5	986809984.4	695057467.3	776250000	546750000	7187.5	5062.5	7187.5	5062.5	7.0927E+12	3.51873E+12	5.5793E+12	2.76792E+12
C		2		40	3,432	3	60	600	137,295	108,000	7187.5	-5062.5	7187.5	-5062.5	986809984.4	-695057467.3	776250000	-546750000	7187.5	-5062.5	7187.5	-5062.5	7.0927E+12	3.51873E+12	5.5793E+12	2.76792E+12
C		3		40	3,432	3	60	600	137,295	108,000	-7187.5	-5062.5	-7187.5	-5062.5	-986809984.4	-695057467.3	-776250000	-546750000	-7187.5	-5062.5	-7187.5	-5062.5	7.0927E+12	3.51873E+12	5.5793E+12	2.76792E+12
C		4		40	3,432	3	60	600	137,295	108,000	-7187.5	5062.5	-7187.5	5062.5	-986809984.4	695057467.3	-776250000	546750000	-7187.5	5062.5	-7187.5	5062.5	7.0927E+12	3.51873E+12	5.5793E+12	2.76792E+12
D	1&4		tD	40	8,500	5	60	600	340,000	180,000	0	5980	0	5660	0	2033200000	0	1018800000	0	5980	0	5660	0	1.21585E+13	0	5.76641E+12
D	2&3			40	8,500	5	60	600	340,000	180,000	0	-5980	0	-5660	0	-2033200000	0	-1018800000	0	-5980	0	-5660	0	1.21585E+13	0	5.76641E+12
E		1	tE	50	3,425	3	55	550	171,250	90,750	8212.5	1975	8212.5	1675	1406390625	338218750	745284375	152006250	8212.5	1975	8212.5	1675	1.155E+13	6.67982E+11	6.12065E+12	2.5461E+11
E		2		50	3,425	3	55	550	171,250	90,750	8212.5	-1975	8212.5	-1675	1406390625	-338218750	745284375	-152006250	8212.5	-1975	8212.5	-1675	1.155E+13	6.67982E+11	6.12065E+12	2.5461E+11
E		3		50	3,425	3	55	550	171,250	90,750	-8212.5	-1975	-8212.5	-1675	-1406390625	-338218750	-745284375	-152006250	-8212.5	-1975	-8212.5	-1675	1.155E+13	6.67982E+11	6.12065E+12	2.5461E+11
E		4		50	3,425	3	55	550	171,250	90,750	-8212.5	1975	-8212.5	1675	-1406390625	338218750	-745284375	152006250	-8212.5	1975	-8212.5	1675	1.155E+13	6.67982E+11	6.12065E+12	2.5461E+11
F		1		35	2,500	1	50	500	87,500	25,000	5250	1982.5	5250	1715	459375000	173468750	131250000	42875000	5250	1982.5	5250	1715	2.41172E+12	3.43902E+11	6.89063E+11	73530625000
F		2		35	2,500	1	50	500	87,500	25,000	5250	-1982.5	5250	-1715	459375000	-173468750	131250000	-42875000	5250	-1982.5	5250	-1715	2.41172E+12	3.43902E+11	6.89063E+11	73530625000
F		3		35	2,500	1	50	500	87,500	25,000	-5250	-1982.5	-5250	-1715	-459375000	-173468750	-131250000	-42875000	-5250	-1982.5	-5250	-1715	2.41172E+12	3.43902E+11	6.89063E+11	73530625000
F		4		35	2,500	1	50	500	87,500	25,000	-5250	1982.5	-5250	1715	-459375000	173468750	-131250000	42875000	-5250	1982.5	-5250	1715	2.41172E+12	3.43902E+11	6.89063E+11	73530625000
G	1&2			30	4,000	2	50	500	120,000	50,000	3985	0	3720	0	478200000	0	186000000	0	3985	0	3720	0	1.90563E+12	0	6.9192E+11	0
G	3&4			30	4,000	2	50	500	120,000	50,000	-3985	0	-3720	0	-478200000	0	-186000000	0	-3985	0	-3720	0	1.90563E+12	0	6.9192E+11	0
H		1		35	3,960	2	50	500	138,600	50,000	3982.5	3980	3715	3980	551974500	551628000	185750000	199000000	3982.5	3980	3715	3980	2.19824E+12	2.19548E+12	6.90061E+11	7.9202E+11
H		2		35	3,960	2	50	500	138,600	50,000	3982.5	-3980	3715	-3980	551974500	-551628000	185750000	-199000000	3982.5	-3980	3715	-3980	2.19824E+12	2.19548E+12	6.90061E+11	7.9202E+11
H		3		35	3,960	2	50	500	138,600	50,000	-3982.5	-3980	-3715	-3980	-551974500	-551628000	-185750000	-199000000	-3982.5	-3980	-3715	-3980	2.19824E+12	2.19548E+12	6.90061E+11	7.9202E+11
H		4		35	3,960	2	50	500	138,600	50,000	-3982.5	3980	-3715	3980	-551974500	551628000	-185750000	199000000	-3982.5	3980	-3715	3980	2.19824E+12	2.19548E+12	6.90061E+11	7.9202E+11
									4,626,001	2,034,700					0	0	0	0					2.3711E+14	6.02886E+13	9.41181E+13	3.0816E+13
									sumA =	6,660,701														Ixx =	3.31228E+14	
																	XNA	0							Iyy =	9.11045E+13
																	YNA	0								

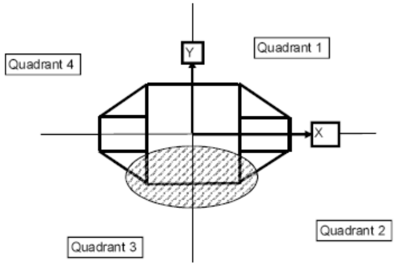


plate	quadrant		tp	bp	ns	ts	bs	Api	Asi	Xopi	Yopi	Xosi	Yosi	Api*Xopi	Api*Yopi	Asi*Xosi	Asi*Yosi	XNApi	YNApi	XNAsi	YNAsi	Api*XNApi^2	Api*YNApi^2	Asi*XNAsi^2	Asi*YNAsi^2
A	1&2	tA	75	4,500	1	75	750	337,500	56,250	9962.5	0	9550	0	3362343750	0	537187500	0	9962.5	-1334.5	9550	-1334.5	3.34973E+13	6.0108E+11	5.13014E+12	1.0018E+11
A	3&4		75	4,500	1	75	750	337,500	56,250	-9962.5	0	-9550	0	-3362343750	0	-537187500	0	-9962.5	-1334.5	-9550	-1334.5	3.34973E+13	6.0108E+11	5.13014E+12	1.0018E+11
B	1	tB	65	3,432	2	68	675	223,105	91,800	9062.5	3187.5	9062.5	3187.5	2021887848	711146760.5	831937500	292612500	9062.5	1852.97	9062.5	1852.97	1.83234E+13	7.6603E+11	7.53943E+12	3.15194E+11
B	2		65	3,432	2	68	675	223,105	91,800	9062.5	-3187.5	9062.5	-3187.5	2021887848	-711146760	831937500	-292612500	9062.5	-4522	9062.5	-4522	1.83234E+13	4.5622E+12	7.53943E+12	1.8772E+12
B	3		65	3,432	2	68	675	223,105	91,800	-9062.5	-3187.5	-9062.5	-3187.5	-2021887848	-711146760	-831937500	-292612500	-9062.5	-4522	-9062.5	-4522	1.83234E+13	4.5622E+12	7.53943E+12	1.8772E+12
B	4		65	3,432	2	68	675	223,105	91,800	-9062.5	3187.5	-9062.5	3187.5	-2021887848	711146760.5	-831937500	292612500	-9062.5	1852.97	-9062.5	1852.97	1.83234E+13	7.6603E+11	7.53943E+12	3.15194E+11
C	1	tC	40	3,432	3	60	600	137,295	108,000	7187.5	5062.5	7187.5	5062.5	986809984.4	695057467.3	776250000	546750000	7187.5	3727.97	7187.5	3727.97	7.0927E+12	1.9081E+12	5.5793E+12	1.50096E+12
C	2		40		3	60		0	0	7187.5	-5062.5	7187.5	-5062.5	0	0	0	0	7187.5	-6397	7187.5	-6397	0	0	0	0
C	3		40		3	60		0	0	-7187.5	-5062.5	-7187.5	-5062.5	0	0	0	0	-7187.5	-6397	-7187.5	-6397	0	0	0	0
C	4		40	3,432	3	60	600	137,295	108,000	-7187.5	5062.5	-7187.5	5062.5	-986809984	695057467.3	-776250000	546750000	-7187.5	3727.97	-7187.5	3727.97	7.0927E+12	1.9081E+12	5.5793E+12	1.50096E+12
D	1&4	tD	40	8,500	5	60	600	340,000	180,000	0	5980	0	5660	0	2033200000	0	1018800000	0	4645.47	0	4325.47	0	7.3373E+12	0	3.36774E+12
D	2&3		40		5	60		0	0	0	-5980	0	-5960	0	0	0	0	0	-7314.5	0	-7294.5	0	0	0	0
E	1	tE	50	3,425	3	55	550	171,250	90,750	8212.5	1975	8212.5	1675	1406390625	338218750	745284375	152006250	8212.5	640.467	8212.5	340.467	1.155E+13	7.0246E+10	6.12065E+12	10519556619
E	2		50	3,425	3	55	550	171,250	90,750	8212.5	-1975	8212.5	-1675	1406390625	-338218750	745284375	-152006250	8212.5	-3309.5	8212.5	-3009.5	1.155E+13	1.8757E+12	6.12065E+12	8.21949E+11
E	3		50	3,425	3	55	550	171,250	90,750	-8212.5	-1975	-8212.5	-1675	-1406390625	-338218750	-745284375	-152006250	-8212.5	-3309.5	-8212.5	-3009.5	1.155E+13	1.8757E+12	6.12065E+12	8.21949E+11
E	4		50	3,425	3	55	550	171,250	90,750	-8212.5	1975	-8212.5	1675	-1406390625	338218750	-745284375	152006250	-8212.5	640.467	-8212.5	340.467	1.155E+13	7.0246E+10	6.12065E+12	10519556619
F	1		35	2,500	1	50	500	87,500	25,000	5250	1982.5	5250	1715	459375000	173468750	131250000	42875000	5250	647.967	5250	380.467	2.41172E+12	3.6738E+10	6.89063E+11	3618884073
F	2		35	2,500	1	50	500	87,500	25,000	5250	-1982.5	5250	-1715	459375000	-173468750	131250000	-42875000	5250	-3317	5250	-3049.5	2.41172E+12	9.6274E+11	6.89063E+11	2.32491E+11
F	3		35	2,500	1	50	500	87,500	25,000	-5250	-1982.5	-5250	-1715	-459375000	-173468750	-131250000	-42875000	-5250	-3317	-5250	-3049.5	2.41172E+12	9.6274E+11	6.89063E+11	2.32491E+11
F	4		35	2,500	1	50	500	87,500	25,000	-5250	1982.5	-5250	1715	-459375000	173468750	-131250000	42875000	-5250	647.967	-5250	380.467	2.41172E+12	3.6738E+10	6.89063E+11	3618884073
G	1&2		30	4,000	2	50	500	120,000	50,000	3985	0	3720	0	478200000	0	186000000	0	3985	-1334.5	3720	-1334.5	1.90563E+12	2.1372E+11	6.9192E+11	89048876975
G	3&4		30	4,000	2	50	500	120,000	50,000	-3985	0	-3720	0	-478200000	0	-186000000	0	-3985	-1334.5	-3720	-1334.5	1.90563E+12	2.1372E+11	6.9192E+11	89048876975
H	1		35	3,960	2	50	500	138,600	50,000	3982.5	3980	3715	3980	551974500	551628000	185750000	199000000	3982.5	2645.47	3715	2645.47	2.19824E+12	9.6999E+11	6.90061E+11	3.49925E+11
H	2		35		2	50		0	0	3982.5	-2000	3965	-2000	0	0	0	0	3982.5	-3334.5	3965	-3334.5	0	0	0	0
H	3		35		2	50		0	0	-3982.5	-2000	-3965	-2000	0	0	0	0	-3982.5	-3334.5	-3965	-3334.5	0	0	0	0
H	4		35	3,960	2	50	500	138,600	50,000	-3982.5	3980	-3715	3980	-551974500	551628000	-185750000	199000000	-3982.5	2645.47	-3715	2645.47	2.19824E+12	9.6999E+11	6.90061E+11	3.49925E+11
								3,734,210	1,538,700					0	4526570935	0	2510300000					2.18528E+14	3.127E+13	8.15794E+13	1.39699E+13
								sumA =	5,272,910														Ixx =	3.00107E+14	
																XNA	0							Iyy =	4.52403E+13
																YNA	1,335								

5.5.2.2 Explosion

INTRODUCTION

PURPOSE

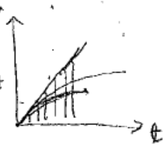
The purpose of this book is to assess the effect of the explosion

INPUT

Spec on page 903

FREE ACCELERATIONS

Assume panels of leg are NOT attached to the structure



Side plates D at segment 4 ⇒ 40 mm PL

Stiffs = 600×60

$\therefore$ Mass/m² = $\left(0.040 + \frac{0.6\times0.06}{1.5}\right)7850 = (0.040 + 0.024)7850 = 502$ kg/m² (0.064)

Max Force = 190 MN/m²

Uniform pressure over period T

Unrestrained plate ⇒ acceleration $a_0 = \frac{\text{force}}{\text{mass}} = \frac{190\times10^6}{502} = 378\times10^3$ m/sec²

Duration $t = 0.0013$ sec $= 1.3\times10^{-3}$ sec

$s = ut + \frac{1}{2}at^2 = 0 + \frac{1}{2}\times378\times10^3\times(1.3\times10^{-3})^2 = \frac{1}{2}\times378\times10^3\times1.69\times10^{-6}$

$= 0.319$ metres. assuming uniform acceleration

Velocity $= at = 378\times10^3\times1.3\times10^{-3} = 491$ m/sec

kg m/sec² = MA = N

$\therefore$ Kinetic energy/m² $= \frac{1}{2}Mv^2 = \frac{1}{2}\times502\times(491)^2 = 60.5\times10^6$ kg m²/sec²/m²

$= 60.5\times10^6$ N-m/m²

Reducing pressure over period T

$\therefore$ With pressure reducing linearly

then acceleration reduces linearly

$\therefore$ Velocity of plate =

$$v = \int_0^T a\,dt = \int_0^T a_0\left(1-\frac{t}{T}\right)dt = a_0\int_0^T dt - \frac{a_0}{T}\int_0^T t\,dt = a_0[t]_0^T - \frac{a_0}{T}\left[\frac{t^2}{2}\right]_0^T = \left[a_0t - \frac{a_0}{T}\frac{t^2}{2}\right]_0^T$$

$$= a_0T - \frac{a_0}{T}\frac{T^2}{2} = a_0T - \frac{a_0T}{2} = \frac{a_0T}{2} = \frac{378\times10^3\times0.0013}{2} = 246\text{ m/sec}$$

$\therefore$ Distance moved

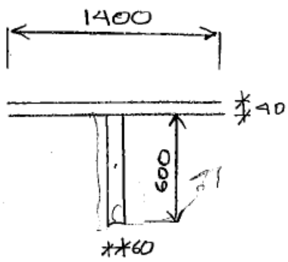
$$s = \int_0^T v\,dt = \int\left(a_0t - \frac{a_0t^2}{2T}\right)dt = a_0\int t\,dt - \frac{a_0}{2T}\int_0^T t^2 = a_0\left[\frac{t^2}{2}\right]_0^T - \frac{a_0}{2T}\left[\frac{t^3}{3}\right]_0^T$$

$$= a_0\frac{T^2}{2} - a_0\frac{T^2}{6} = \frac{a_0T^2}{3} = \frac{378\times10^3\times(1.3)^2\times10^{-6}}{3} = 0.213\text{ metres}$$

& Kinetic energy/m² $= \frac{1}{2}mv^2 = \frac{1}{2}\times502\times(246)^2 = 15.2\times10^6$ kg m²/sec²/m²

15.2×10^6 = N-m/m²

ENERGY IN PLASTIC ROTATIONS



Find plastic modulus W_{pl}

Area of skin = 1400 × 40 = 56000 mm²
stiffener = 600 × 60 = 36000
92 000 mm²

$\frac{A}{2} = \frac{92000}{2} = 46000 \text{ mm}^2$

∴ Plastic neutral axis in plate at $\frac{46000}{1400} = 32.9$ mm

∴ $W_{pl} \doteq 600 \times 60 \, (300 + 32.9) = 11.94 \times 10^6 \text{ mm}^3$
332.9

∴ M_p/stiffener $= \frac{f_y}{\gamma_M} \, 11.94 \times 10^6 \times 460 = 5.49 \times 10^9$ N-mm $= 5.49 \times 10^6$ N-m

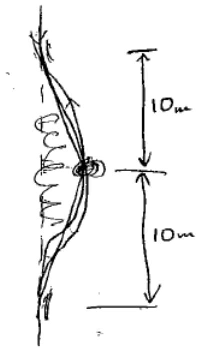
use $\gamma_M = 1.0$ for accident

Kinetic energy per 1.4m width × 10 m height $1.4 \times 10 \times 15.2 \times 10^6 = 213 \times 10^6$ N-m

∴ Rotation required to absorb energy $= \frac{213 \times 10^6}{5.49 \times 10^9} = \frac{213}{5490} = 0.039$ rads

$= 0.039 \times \frac{180}{\pi} = 2.23°$

Assuming 20m damage height
0.039 × 10m = 0.39 metres



6 Transverse Stiffener and Diaphragm Plate Buckling Analysis / Longitudinal to Transverse Stiffener Connections

6.1 Introduction

This document summarizes the following three detailed finite element (FE) analyses performed to support the design of the tower leg transverse stiffening elements:

- Analyses to justify the removal of tab plates connecting longitudinal stiffeners to transverse stiffeners (Task 1);
- Analyses to confirm the stability of the transverse stiffener flanges without intermediate restraint plates bracing the flanges to longitudinal stiffeners (Task 2); and
- Analyses to confirm the stability of the triangular plate diaphragms (Task 3).

The background of each task, description of the models and loadings, and summary of the results are described in the following sections.

6.2 Task 1: Removal of Tab Plates

6.2.1 Background

The general concept submission included tab plates connecting the tower leg longitudinal stiffeners to the webs of every transverse stiffener, as shown in Figure 6-1. Following the general concept submission, the removal of tab plates was investigated to reduce fabrication costs. This removal was considered feasible as it was successfully implemented on the Akashi suspension bridge, based on both experimental and analytical work.

Tab plates typically have three functions:

- To provide lateral restraint to the longitudinal stiffener, if required;
- To improve the stability of the transverse stiffeners; and
- To transfer deviation forces (“kick” forces from buckling of the adjacent compressed panels) from the longitudinal stiffener to the transverse stiffener;

The tab plates are not required to provide lateral restraint to the longitudinal stiffeners, as they are been proportioned to satisfy the maximum width-to-thickness ratios for Class 3 in EN 1993-1-1 Table 5.2 and the requirement of EN 1993-1-5 Section 9.2.1(8). The tab plates are also not required to improve the stability of the transverse stiffeners as the stiffeners are proportioned in such a way that no intermediate restraint is required for the full 8 m span.

Therefore, the tab plates are only required for transferring the deviation forces from the longitudinal stiffeners to the transverse stiffener. The deviation forces are those developed as the longitudinal stiffener kicks out-of-plan and is restrained by the transverse stiffener. Without tab plates, these forces must be carried by the welds between the transverse stiffener web and the skin plate and by the welds between the longitudinal stiffener and the skin plate.

The finite element analyses was used to assess the demands on these welds.

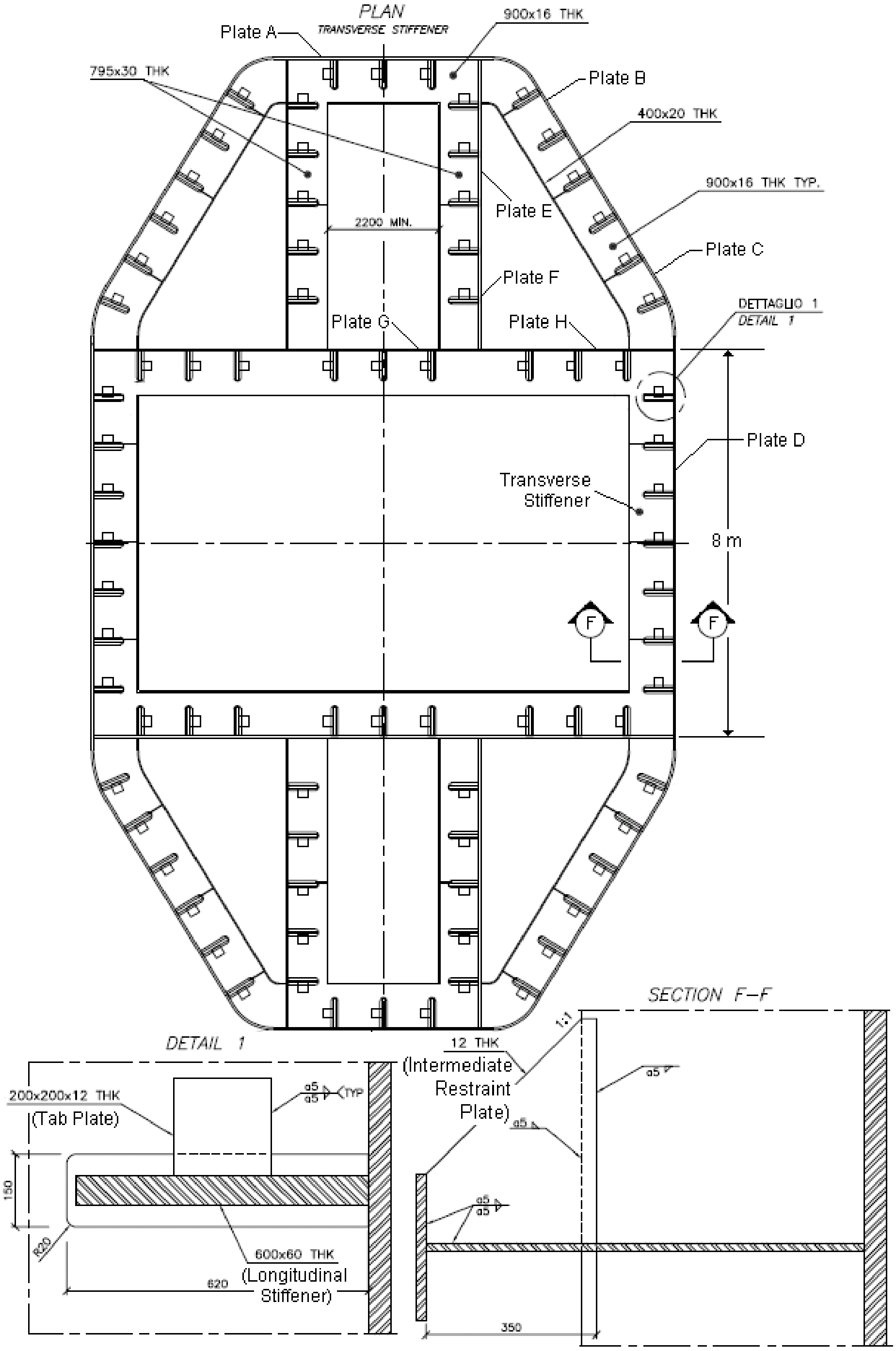


Figure 6-1: Typical tower leg cross section at a transverse stiffener, tab plate details and intermediate restraint plate details.

6.2.2 Model Description

As shown in Figure 6-2, the analysis model for this task comprises:

- A transverse stiffener (denoted as TStiff) with cut-outs 150 mm wide to allow the longitudinal stiffeners (denoted as LStiff) to pass through. The transverse stiffener spans 8.0 m between plates H, ignoring the presence of the longitudinal stiffeners on plates H, as these cannot be relied upon to provide stability as the loading on the tower leg approaches the ultimate limit state, being fully stressed from longitudinal loads;
- The skin plate (denoted as Panel D) extending 3.5 m above and below the transverse stiffener;
- The longitudinal stiffeners extending 3.5 m above and below; and
- The adjoining plate H (denoted as Panel H) extending 3.5 m above and below.

The shell element model was developed using the analysis program SAP2000. The 2-D shell elements are based on the three- or four-node formulation, include both in- and out-of-plane responses, and have isotropic material properties with a Young's modulus of 200,000 MPa and Poisson's ratio of 0.3. The out-of-plane response is based on the Mindlin-Reissner formulation, which includes the effects of shear deformations.

As shown in Figure 6-2, the element meshing was selected and refined to maintain the element aspect ratios close to unity and to minimize the element distortion from the standard rectangular form. Edge constraints were used to provide transitions from coarser mesh to finer mesh. The boundary conditions are also shown in Figure 6-2.

The model does not include the intermediate restraint plates connecting the longitudinal stiffeners to the transverse stiffener flanges, shown in Section F-F of Figure 6-1. Therefore, the same model could also be used for investigating of intermediate restraint removal, as described in Section 6.1, with minimal modification.

Three models were needed to cover all plate conditions:

1 A model with the biggest expected longitudinal stiffeners (Model 01f), as this would develop the largest restraint forces;

2 A model with a thin skin plate (Model 01g), as this might produce higher stress concentrations because of the low flexural stiffness of the plate; and

3 A model with an intermediate skin plate thickness (Model 01h) to ensure that this did not present an unexpectedly demanding condition.

Table 6-1 shows the plate sizes for the three cases.

All the analysis cases were linear elastic.

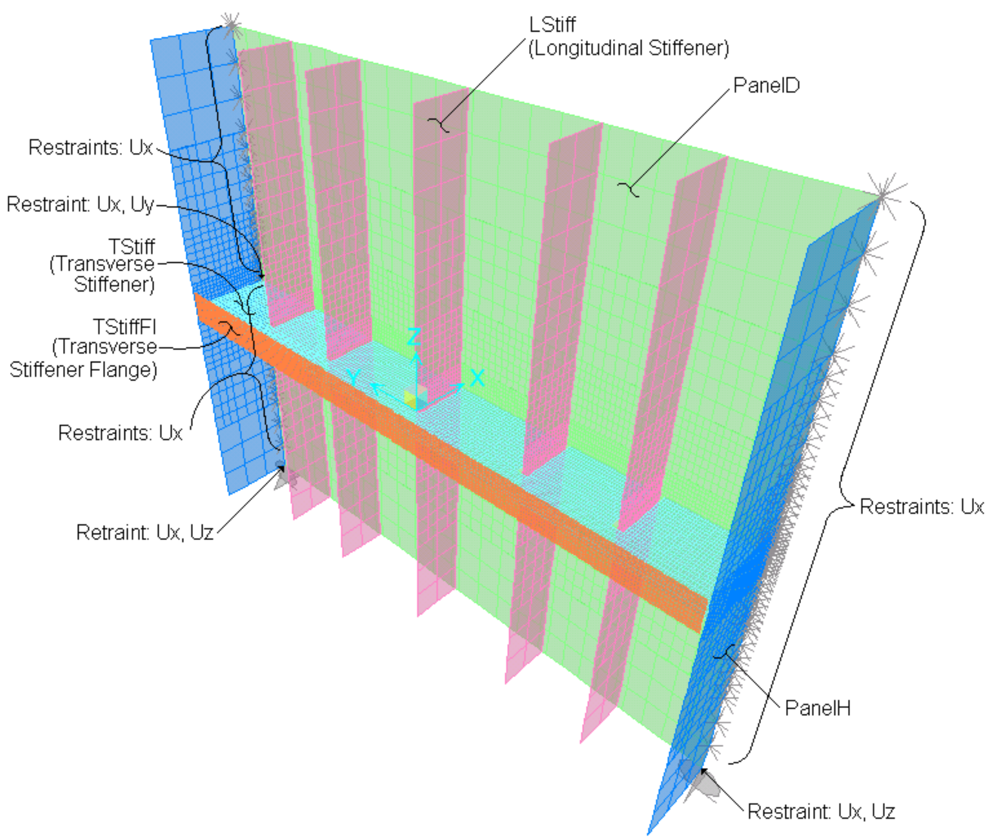


Figure 6-2: Model for tab plate removal investigation.

Case	Panel D Thickness (mm)	LStiff Thickness (mm) x Width (mm)	TStiff Thickness (mm) x Width (mm)	TStiffFl Thickness (mm) x Width (mm)	Panel H Thickness (mm)
01f	85	70x700	16x1090	25x420	35
01g	55	63x625	16x1090	20x390	35
01h	45	63x625	16x1090	20x360	35

Table 6-1: Plate sizes for models used for investigating tab plate removal.

6.2.3 Loadings

The loading for each analysis case was the second-order loading derived from EN 1993-1-5 Section 9.2.1 with the stress in the transverse stiffener limited to that required for stability of the transverse stiffener flange. As shown in Figure 6-3, the loadings are of sinusoidal type with the maximum load per unit length given in Table 6-2.

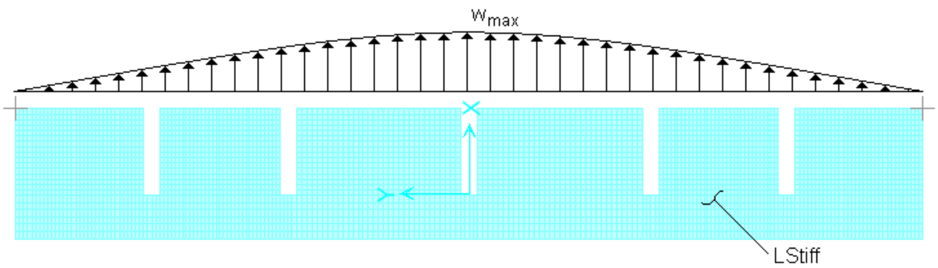


Figure 6-3: Model loading.

Case	w_{max} (kN/m)
01f	479
01g	315
01h	270

Table 6-2: Loading magnitude.

6.2.4 Analysis Results Summary

Figure 6-4, Figure 6-5, and Figure 6-6 show the direct and shear forces cases for 01f, 01g, and 01h, respectively. The sign convention is also shown in each figure.

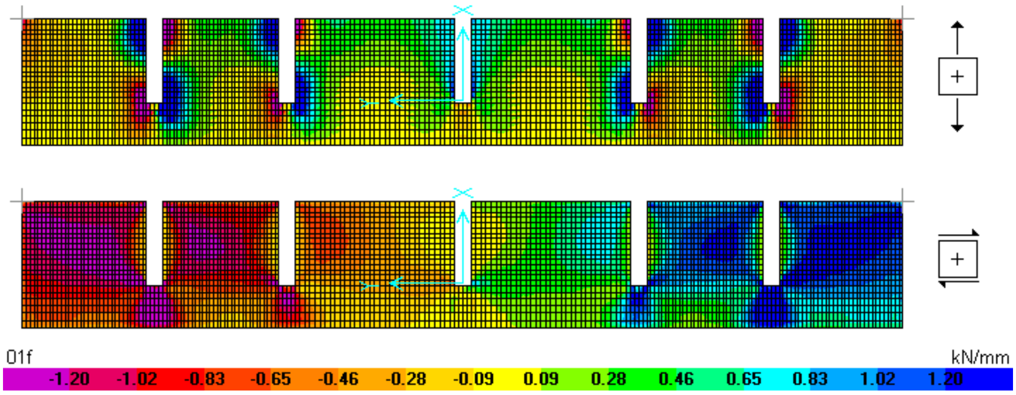


Figure 6-4: Direct and shear forces in transverse stiffener for case 01f.

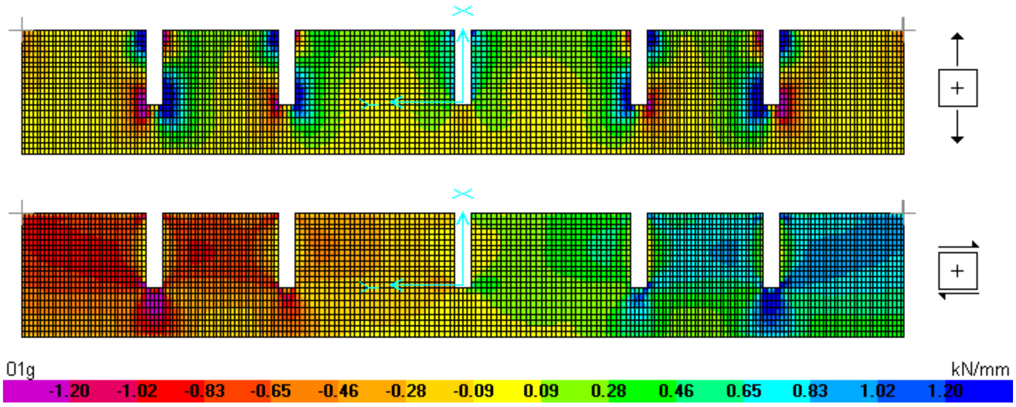


Figure 6-5: Direct and shear forces in transverse stiffener for case 01g.

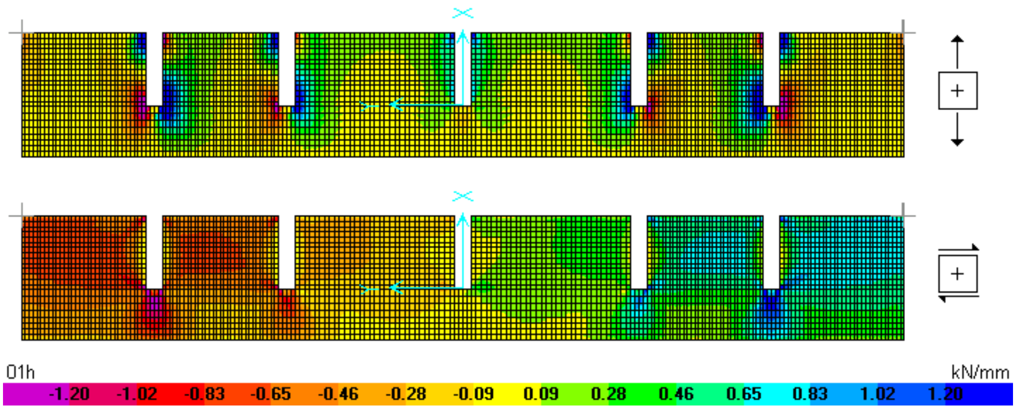


Figure 6-6: Direct and shear forces in transverse stiffener for case 01h.

6.2.5 Verification of Resistance

The analyses give high stresses because they do not allow for yielding of the welds. The welds were checked by summing the forces on an appropriate length and comparing the average load/unit-length with the resistance of the fillet welds calculated in accordance with EN 1993-1-8 Section 4.5.3.2. The length used for averaging the force was 2.5 times skin plate thickness. The value of 2.5 was derived from EN 1993-1-8 Section 4.10 which uses an equivalent angle of dispersion of 1:3.5 on each side of a connection, as shown in Figure 4.8. Because this connection is not exactly the same as shown in Figure 4.8, the value of 3.5 was reduced to 2.5 to account for uncertainty.

For 5mm fillet welds both sides of the web, the utilization ratio was found to be 0.82. Therefore, the tab-plates can be removed from the transverse stiffeners where:

- the longitudinal stiffeners are straight (i.e., no angle change along the longitudinal stiffener); and
- the only loads applied are the deviation forces.

6.3 Task 2: Stability of Transverse Stiffener Flanges

6.3.1 Background

In addition to tab plates connecting the longitudinal stiffeners to the transverse stiffeners, the general concept submission also included restraint plates bracing the transverse stiffener flange to the longitudinal stiffeners, as shown Figure 6-1. These plates were required to provide the minimum weight of flange that satisfied EN 1993-1-5 Section 9.2.1, either by sub-clause (8) or (9).

The removal of the restraints plates was also investigated to reduce fabrication costs. FE analysis was used to find the optimum flange to satisfy EN 1993-1-5 Section 9.2.1(8) or (9) because it accounts for the stress gradient along the transverse stiffener web, producing greater economy than is possible with simple hand calculations.

Following the general concept submission, the transverse stiffener arrangement was changed to simplify fabrication and assembly. Plate A was thickened so that transverse stiffeners are not required for the stability of a single longitudinal stiffener. The transverse stiffeners to plates B, C, E, F and H were replaced by a triangular diaphragm filling the enclosed cell. Therefore, the only tower leg plates D and G require transverse stiffeners. The width of plate G is only 4 m, and so the transverse stiffeners were proportioned to provide increased robustness than would have resulted from the stiffener proportioned considering only the applied loads. Therefore, the focus of the analyses was on the plate D stiffeners, which have a clear span between plates H of 8 m.

6.3.2 Methodology for Sizing Transverse Stiffener Flanges

Hand calculations, confirmed by initial finite element results with a range of flange cross-sections, indicated that the requirements of EN 1993-1-5 Section 9.2.1(8) or (9) are impractical for an 8 m clear span. The requirements of EN 1993-1-5 Section 9.2.1(8) and (9) were then reviewed in the light of traditional design practice, which is to check the stiffener resistance as if the flange and half the web were a strut.

The review shows that requirements of EN 1993-1-5 Section 9.2.1(8) and (9) are reasonable for “stocky” stiffeners (heavy in proportion to their length), but are unreasonable for long stiffeners. This is demonstrated by the recommended value of θ = 6, which corresponds to a relative slenderness of:

$$\bar{\lambda} = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{f_y}{\theta f_y}} = \sqrt{\frac{1}{6}} = 0.41$$

As the slenderness of a strut is permitted to exceed $\bar{\lambda} = 0.41$, the stability verification was changed to the verification of compression resistance from EN 1993-1-1 section 6.3.1 using the elastic buckling stress, σ_{cr}, in the calculation of relative slenderness as per Equations 6.50 and 6.53:

$$\bar{\lambda} = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{Af_y}{N_{cr}}}$$

For a given flange size, the elastic buckling factor and stress can be obtained from an FE analysis, in which the model is loaded according to EN 1993-1-5 Section 9.2.1, but limited so that the maximum stress does not exceed the resistance derived by the initial calculations.

6.3.3 Model and Loading Description

The FE model and loading for obtaining the elastic buckling factors and stresses is essentially the same as that used for Task 1, except that a row of elements of extremely low flexural stiffness was provided along web at the skin plate interface, as shown in Figure 6-7, to respect the requirement of EN 1993-1-5 Section 9.2.1(9) “not considering rotational restraint from the plate.”

The same analysis cases as used in Task 1 and shown in Table 6-1 were also used to confirm the required transverse stiffener flange sizes. The transverse stiffener flange sizes used for each model case are: 420×25 for case 01f, 390×20 mm for case 01g, and 360×20 for case 01h.

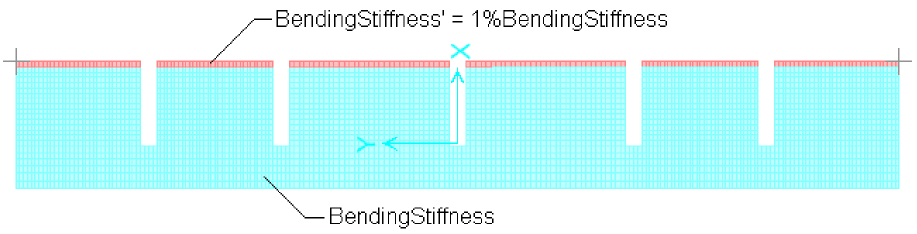


Figure 6-7: Model for checking stability of transverse stiffener flange.

6.3.4 Analysis Result Summary

The elastic buckling factor and the elastic buckling stress, σ_{cr}, for each flange size are given in Table 6-3. The corresponding buckled shapes are shown in Figure 6-8, Figure 6-9 and Figure 6-10, for case 01f, 01g and 01h, respectively.

The presented elastic buckling factors are those associated with the lowest (1st) buckling modes. The buckling load is the product of the elastic buckling factor and the applied loading.

Case	**Elastic Buckling Factor**	σ_{cr} **(MPa)**
01f	2.31	2.31(171) = 395
01g	2.56	2.56(140) = 358
01h	2.59	2.59(127) = 329

Table 6-3: Elastic buckling factors and stresses.

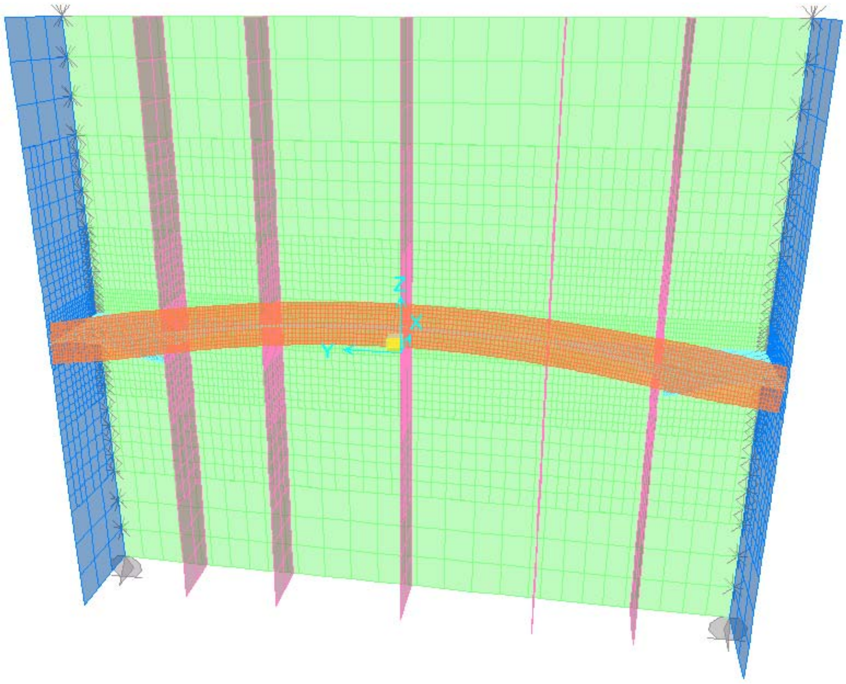


Figure 6-8: 1st Mode Buckling Shape for Case 01f.

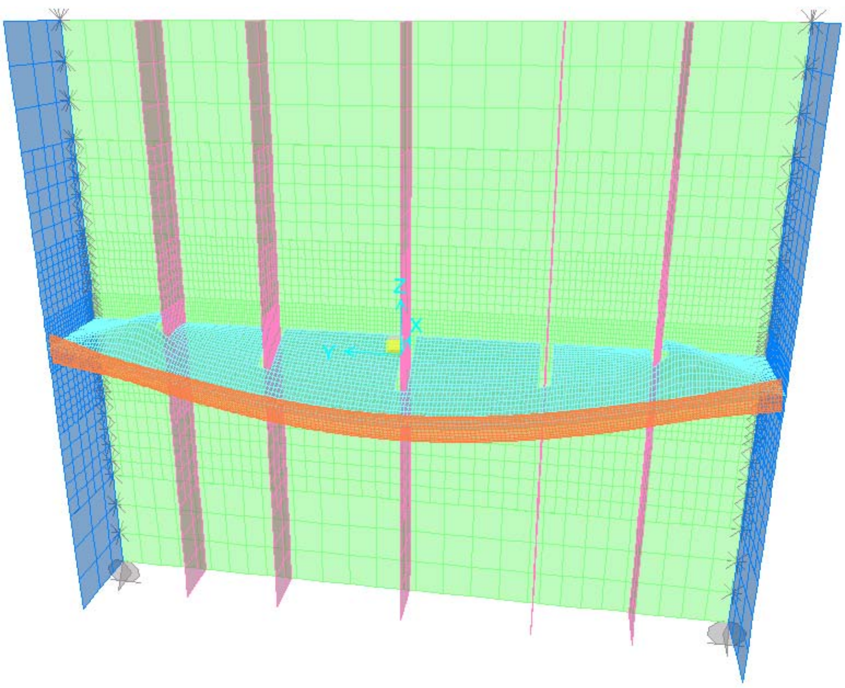


Figure 6-9: 1st Mode Buckling Shape for Case 01g.

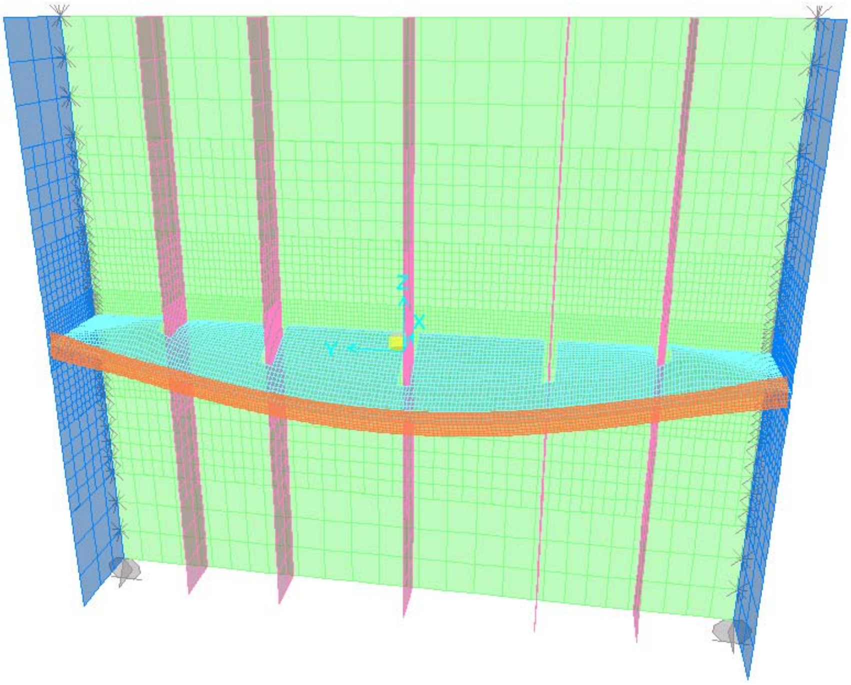


Figure 6-10: 1st Mode Buckling Shape for Case 01h.

6.3.5 Verification of Resistance

The stability verification uses the maximum stress, $\frac{\chi f_y}{\gamma_{M1}}$, from EN 1993-1-1 Section 6.3.1, but the calculation of χ is based on the increased imperfection factor, α_e, from EN 1993-1-5 Section 4.5.3(5), to allow for the greater fabrication tolerance for the stiffener flanges compared with typical column flanges. The relative slenderness is calculated using elastic buckling stress from Table 6-3 in the equation:

$$\bar{\lambda} = \sqrt{\frac{f_y}{\sigma_{cr}}} = \sqrt{\frac{Af_y}{N_{cr}}}$$ (equations 6.50 and 6.53 of EN 1993-1-1)

This stress is used as the limiting stress in EN 1993-1-5 Section 9.2.1 to check at which locations each flange is appropriate.

6.4 Task 3: Stability of Triangular Diaphragm Plates

6.4.1 Background

Following the general concept submission, the transverse stiffener arrangement was changed to simplify fabrication. The transverse stiffeners to plates B, C, E, F and H were replaced by a triangular diaphragm filling the enclosed cell. The stability of this diaphragm was checked by EN 1993-1-5 Section 10.

The FE model is used to assess the minimum load amplifier, α_{cr}, for the design loads to reach the elastic critical load and the maximum von Mises stress on the plate from a linear analysis of the applied loads. The diaphragm model used to determine these parameters is described below.

6.4.2 Model Description

As shown in Figure 6-11, the model for this task comprises:

- A flat plate diaphragm with cut-outs (denoted TStiff) to allow the longitudinal stiffeners to pass through, and a circular hole of 1.5 m diameter;
- The adjoining plates B, C, E and F (denoted Panel B, Panel C, Panel E, and Panel F, respectively) extending 3.5 m above and below the diaphragm;
- The longitudinal stiffeners for plates B, C, E, and F (denoted LStiffB, LStiffC, LStiffE, and LStiffE, respectively) extending 3.5 m above and below; and
- The ring plate (denoted TSitffR) around the circular hole.

For simplicity, plate F was defined to be the same as plate E, with the combined plate denoted Panel E. Likewise, the longitudinal stiffeners for plate F were defined to be the same as those for plate E, denoted as LStiffE. The effect of the triangular diaphragm stability is insignificant because the thickness of plate F is so much larger than that of the diaphragm. The plate sizes are given in Table 6-4.

Similar to the models created for Tasks 1 and 2, the shell element model for this task was developed using SAP2000. The 2-D shell elements are based on the three- or four-node formulation, and include both in- and out-of-plane responses, and have isotropic material properties with Young’s modulus of 200,000 MPa and Poisson’s ratio of 0.3. The out-of-plane response is based on the Mindlin-Reissner formulation, which includes the effects of shear deformations. The element meshing was selected and refined to maintain the element aspect ratios close to unity and to minimize the element distortion from the standard rectangular form. Edge constraints were used to provide transition from coarser mesh to finer mesh. The boundary conditions are shown in Figure 6-11.

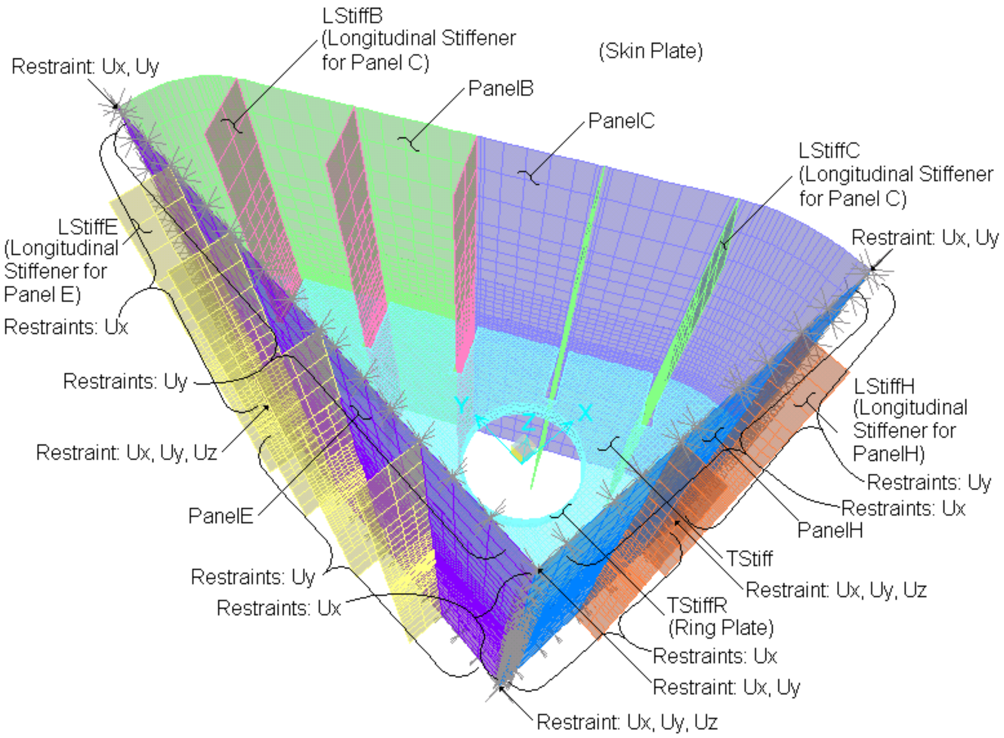


Figure 6-11: Model for triangular diaphragm stability check.

Panel B Thickness (mm)	100
Panel C Thickness (mm)	100
Panel E Thickness (mm)	100
Panel H Thickness (mm)	35
TStiff Thickness (mm)	20
LStiffB Thickness (mm) x Width (mm)	70x700
LStiffC Thickness x Width (mm)	70x700
LStiffE Thickness x Width (mm)	60x600
LStiffH Thickness x Width (mm)	48x475
TStiffR Thickness (mm) x Width (mm)	20x150

Table 6-4: Plate sizes

.

6.4.3 Loadings

The loading for each plate bounding the triangular diaphragm was the second-order in-plane loading calculated from EN 1993-1-5 Section 9.2.1, but assuming no growth of imperfection because of the in-plane diaphragm stiffness. As shown in Figure 6-12, the loadings are of sinusoidal type with the maximum load per unit length given in Table 6-5. To account for all buckling mode possibilities, the loading directions were permutated, as indicated in Table 6-6.

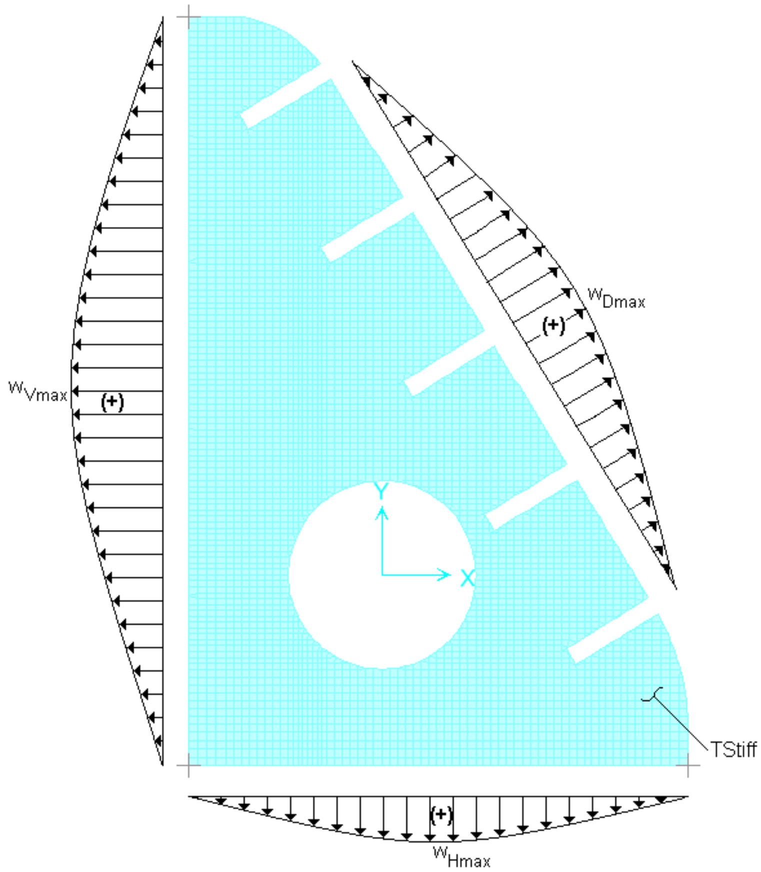


Figure 6-12: Diaphragm plate loading.

Loading	Peak Value (kN/m)
w_D	567
w_H	160
w_V	562

Table 6-5: Diaphragm plate loading magnitude.

Case	w_D	w_H	w_V
a	+	+	+
b	+	-	+
c	+	+	-
d	+	-	-
e	-	-	-
f	-	+	-
g	-	-	+
h	-	+	+

Table 6-6: Diaphragm plate loading directions.

6.4.4 Analysis Result Summary

The 1st buckling modes for cases a to f are shown in Figure 6-13 to Figure 6-20, respectively. In all cases, Panel E, Panel H, LStiffE, LStiffH elements are not shown for clarity.

The elastic buckling factor and maximum von Mises stresses for each case are shown in Table 6-7. The presented elastic buckling factors are those associated with the lowest (1st) buckling modes. The buckling load is the product of the elastic buckling factor and the applied loading. A negative elastic buckling factor indicates that the buckling loading occurs if the loading directions are reversed.

Case	Elastic Buckling Factor	$\sigma_{vonMises}$ (MPa)
a	-2.17	46
b	-1.98	58
c	-2.68	252
d	-3.34	212
e	2.17	46
f	1.98	58
g	2.68	252
h	3.34	212

Table 6-7: Plate diaphragm elastic buckling factors and critical von Mises stresses.

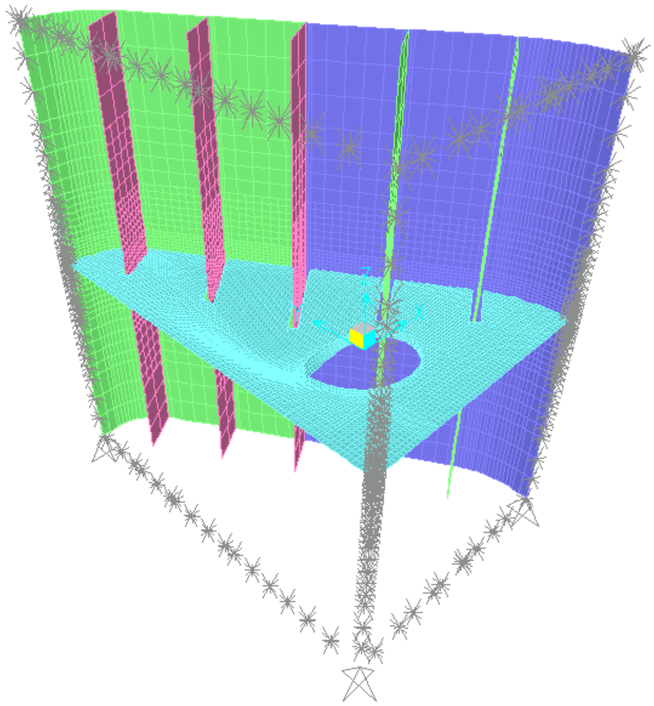


Figure 6-13: 1st Mode Buckling Shape for Case a.

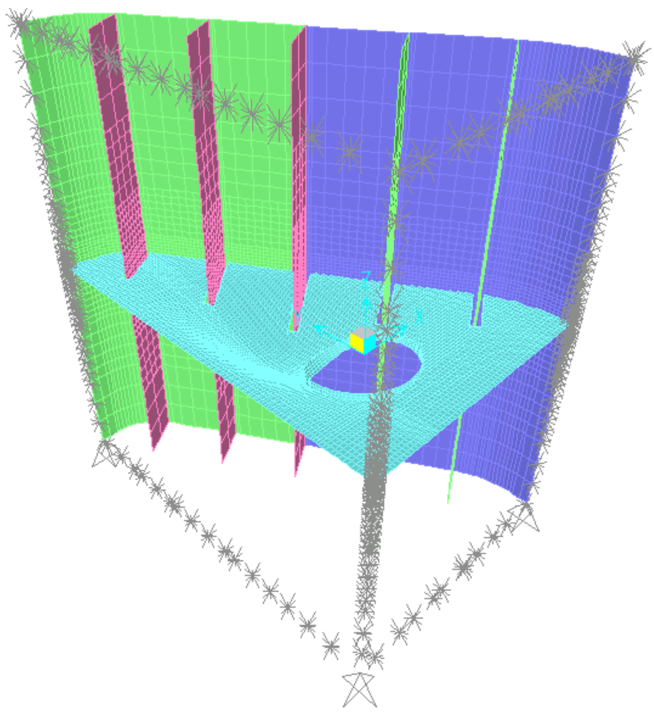


Figure 6-14: 1st Mode Buckling Shape for Case b.

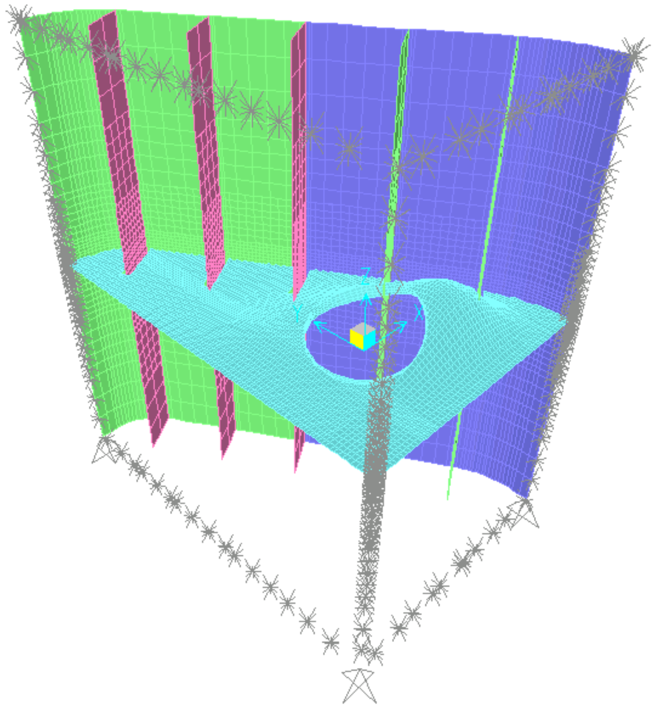


Figure 6-15: 1st Mode Buckling Shape for Case c.

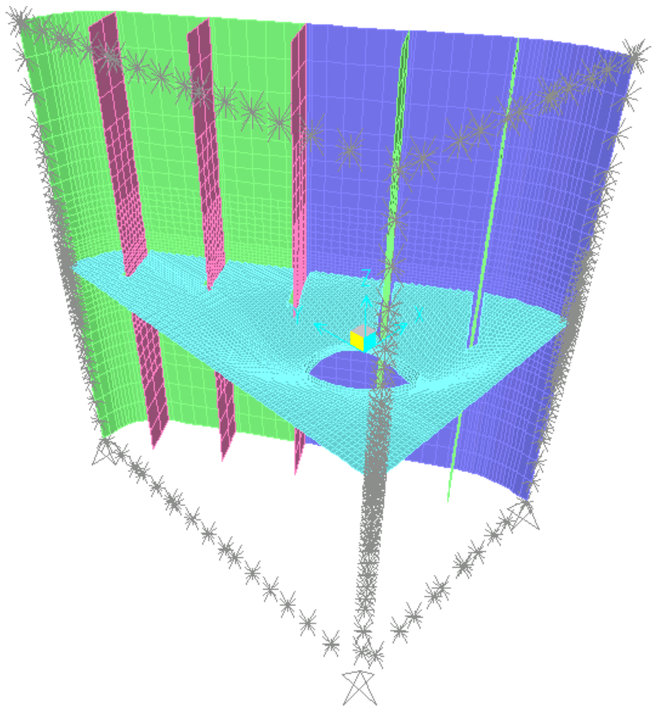


Figure 6-16: 1st Mode Buckling Shape for Case d.

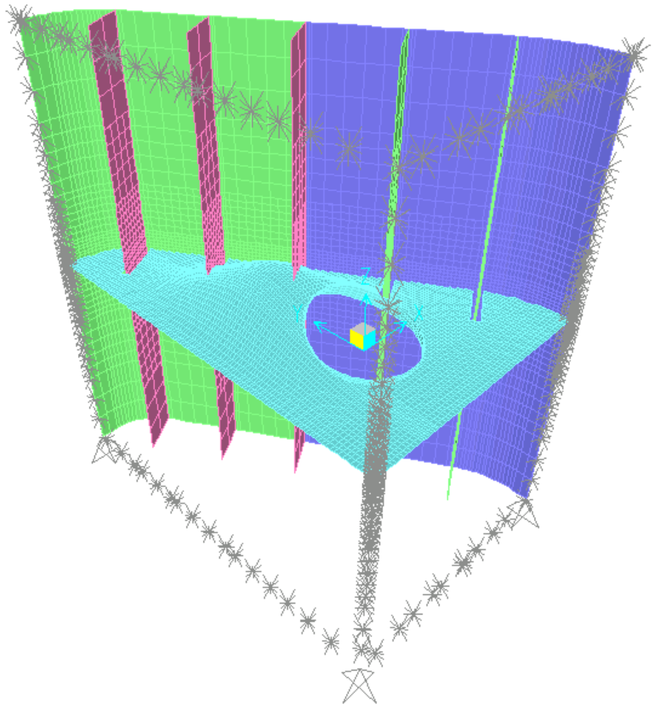


Figure 6-17: 1st Mode Buckling Shape for Case e.

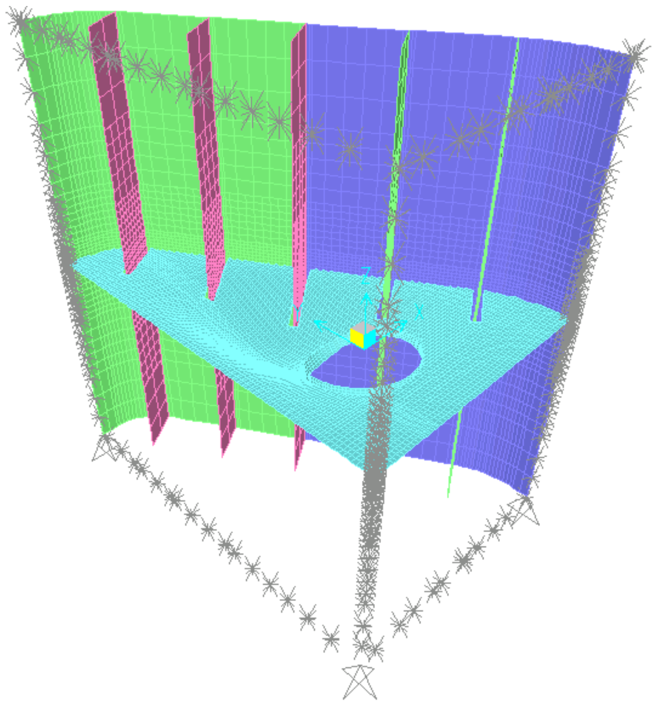


Figure 6-18: 1st Mode Buckling Shape for Case f.

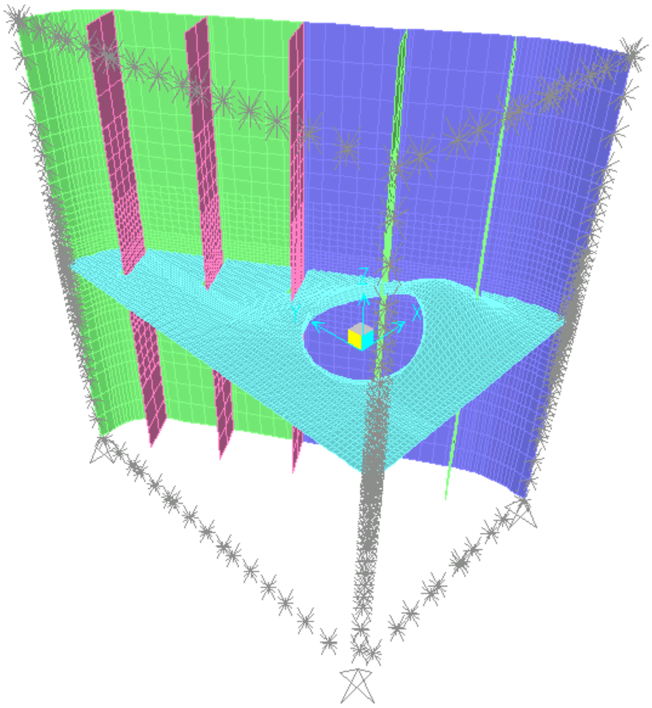


Figure 6-19: 1st Mode Buckling Shape for Case g.

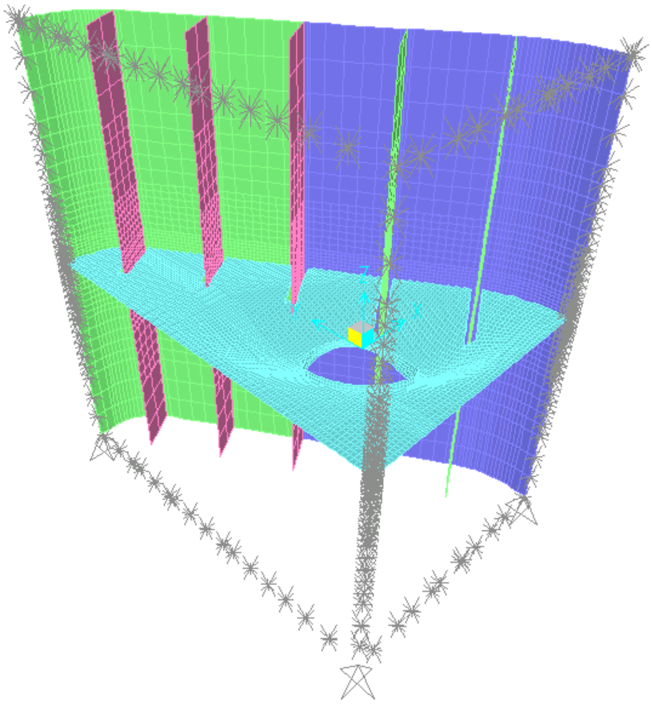


Figure 6-20: 1st Mode Buckling Shape for Case h.

6.4.5 Verification of Resistance

The diaphragm stability was checked by EN 1993-1-5 Section 10 and Annex B.1. The relative slenderness of the plate is calculated using EN 1993-1-5 equation (10.2), in which:

- α_{cr} is the buckling factor from the FE model run for the elastic buckling solution; and
- $\alpha_{ult,k}$ is the load amplifier for design loads to reach the characteristic value of resistance, which may be taken as the ratio of the max von Mises stress to the yield stress, where the von Mises stresses are obtained from a linear FE analysis.

The resistance equation is equation (10.2) of EN 1993-1-5 in which ρ is found from equation B.1 of Annex B.1 of EN 1993-1-5.

The above calculation gives a utilization ratio of 0.97. The effects of potential out-of-plane loading are accommodated by adding the ring stiffener around the hole and the transverse 160×16 stiffener under the diaphragm.