Stretto di Messina

Concessionaria per la progettazione, realizzazione e gestione del collegamento stabile tra la Sicilia
e il Continente Organismo di Diritto Pubblico
(Legge n° 1158 del 17 dicembre 1971, modificata dal D.Lgs. n°114 del 24 aprile 2003)

EurolinK

PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

EUROLINK S.C.p.A.

IMPREGILO S.p.A. (MANDATARIA)
SOCIETÀ ITALIANA PER CONDOTTE D'ACQUA S.p.A. (MANDANTE)
COOPERATIVA MURATORI E CEMENTISTI - C.M.C. DI RAVENNA SOC. COOP. A.R.L. (MANDANTE)
SACYR S.A.U. (MANDANTE)
ISHIKAWAJIMA - HARIMA HEAVY INDUSTRIES CO. LTD (MANDANTE)
A.C.I. S.C.P.A. - CONSORZIO STABILE (MANDANTE)

IL PROGETTISTA	IL CONTRAENTE GENERALE	STRETTO DI MESSINA	STRETTO DI MESSINA
Ing E.M.Veje COWI EurolinK Dott. Ing. E. Pagani Ordine Ingegneri Milano n° 15408	Project Manager (Ing. P.P. Marcheselli)	Direttore Generale e RUP Validazione (Ing. G. Fiammenghi)	Amministratore Delegato (Dott. P. Ciucci)

PS0043_F0

Unità Funzionale	OPERA DI ATTRAVERSAMENTO
Tipo di sistema	SOVRASTRUTTURE
Raggruppamento di opere/attività	SISTEMA DI SOSPENSIONE
Opera - tratto d'opera - parte d'opera	Cable System
Titolo del documento	Design Report - Main Cables and Anchors

CODICE | C | G | 1 | 0 | 0 | 0 | | P | | C | L | | D | | P | | S | V | | S | 7 | | S | S | | 0 | 0 | | 0 | 0 | | 0 | 0 | | 0 | 1 | | F0 |

REV	DATA	DESCRIZIONE	REDATTO	VERIFICATO	APPROVATO
F0	20/06/2011	EMISSIONE FINALE	CJW	IPTF	CJW/LSJ

NOME DEL FILE: PS0043_F0

INDICE

1 Introduction

This report describes the design of the following structural elements of the cable system:

- Main cables
- Anchorage crossheads and anchor bars

The design is based on the design shown in the Tender Design.

For some items it is found advantageous to introduce changes to the design and the following changes are introduced:

- The main cable strand arrangement has been altered from a matrix arrangement to a vertical staggered arrangement to improve stability of the cable prior to compaction and to improve constructability.
- The main cable wire diameter has been increased from 5.32 mm to 5.40 mm and the number of strands in main span is increased from 324 to 349. Both changes have been made to accommodate increases in deck weight. The number of additional strands in the side spans has increased from 8 and 6 in Sicilia and Calabria span respectively to 12 and 8. This has resulted in an increase in cross-sectional area compared to the Tender design of:10.6 % in the main span and 11.6 % / 11.0 % in the Sicilia / Calabria side spans respectively.
- Conventional round wire-wrapping has been added to the main cable. This will be applied without paste beneath the Cableguard elastomeric wrapping system.
- The main cable spacing has increased from 1750 to 2000 mm due to the revised strand arrangement at the saddles.
- The arrangement of PPWS strands at the anchorage crossheads has been extensively revised to suit a new anchorage system consisting of looped PT tendons rather than Dwyidag bars.
- Anchorage crossheads have been redesigned and optimised to suit the new arrangement and to improve access for installing the anchor bars during construction.

- The vertical spacing of crossheads at the anchor wall has been increased to reduce congestion of the PPWS sockets and improve tolerance of the arrangement to PPWS strand length.

All the calculations are based on the global IBDAS model version 3.3f unless otherwise noted.

2 Design References

References relevant to the elements of the cable system described within this report are given below:

2.1 Design Specifications

CG.10.00-P-RG-D-P-GE-00-00-00-00-00-02-B - "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 1993-1-9	Design of Steel Structures – Part 1-9: Fatigue
EN 1993-1-11	Design of steel structures - Part 1-11: Design of structures with tension components
EN 1993-2	Design of Steel Structures – Part 2: Steel Bridges
RFI No. 44F	Rete Ferroviaria Italia - Istruzione No. 44F "Verifiche a fatica dei ponti ferroviari"
PTI DC45.1-07	Recommendations for stay cable design, testing and installation, 5th Edition
fib Bulletin 30	Acceptance of stay cable systems using prestressing steels, 2005
SETRA	CIP recommendations on cable stays

2.3 Material Specifications

BS 1052:1980	Specification for mild steel wire for general engineering purposes
EN 1774:1997	Zinc and zinc alloys: Alloys for foundry purposes - Ingot and liquid

EN 10083-1:2006	Steels for quenching and tempering - General technical delivery conditions
EN 10083-2:2006	Steels for quenching and tempering - Technical delivery conditions for non-alloy steels
EN 10083-3:2006	Steels for quenching and tempering - Technical delivery conditions for alloy steels
EN 10244-1:2009	Steel wire and wire products. Non-ferrous metallic coatings on steel wire. General principles
EN 10244-2:2009	Steel wire and wire products. Non-ferrous metallic coatings on steel wire. Zinc or zinc alloy coatings
EN 10264-1:2002	Steel wire and wire products. Steel wire for ropes.
EN 10264-2:2002	Steel wire and wire products. Steel wire for ropes. Cold drawn non alloy steel wire for ropes for general applications
EN 10264-3:2002	Steel wire and wire products. Steel wire for ropes. Round and shaped non alloyed steel wire for high duty applications
EN 10277-1:2008	Bright steel products. Technical delivery conditions - General
EN 10277-2:2008	Bright steel products. Technical delivery conditions - Steels for general engineering purposes
EN 10227-3:2008	Bright steel products. Technical delivery conditions - Free-cutting steels
EN 10277-5:2008	Bright steel products. Technical delivery conditions - Steels for quenching and tempering
EN 12385-1:2002	Steel wire ropes - Safety - Part 1: General requirements (+A1:2008)
EN 12385-2:2002	Steel wire ropes - Safety - Part 2: Definitions, designation and classification (+A1:2008)
EN 12385-3:2002	Steel wire ropes - Safety - Part 3: Information for use and maintenance (+A1:2008)
EN 12385-10:2002	Steel wire ropes - Safety - Part 10: Spiral ropes for general structural applications (+A1:2008)

EN 13411-4:2002 Terminations for steel wire ropes - Safety. Metal and resin socketing (+ A1:2008)

EN 20898-1:1992 Mechanical properties of fasteners - Part 1: Bolts, screws and studs (ISO 898-1:1988)

EN 20898-2:1994 Mechanical properties of fasteners – Part 2: Nuts with specified proof load values – Coarse thread (ISO 898-2:1992).

EN 22063:1995 Metallic and other inorganic coatings. Thermal spraying Zinc, aluminium and their alloys.

2.4 Drawings

The cable system design drawings relevant for this report are listed in Table 2-1.

Table 2-1 Cable systems drawings relevant for this report

Drawing Title	Drawing Number
General arrangement	CG1000-P-AX-D-P-SV-S7-SS-00-00-00-01
Main cable / Strand arrangement	CG1000-P-AX-D-P-SV-S7-CA-00-00-00-01
Main cable / Geometry	CG1000-P-AX-D-P-SV-S7-CA-00-00-00-02
Main cable / Anchorages - Sicilia - General arrangement	CG1000-P-AX-D-P-SV-S7-CA-00-00-00-03
Main cable / Anchorages - Calabria - General arrangement	CG1000-P-AX-D-P-SV-S7-CA-00-00-00-04
Main cable / Anchorages - Details 1	CG1000-P-BX-D-P-SV-S7-CA-00-00-00-01
Main cable / Anchorages - Details 2	CG1000-P-BX-D-P-SV-S7-CA-00-00-00-02

3 Nomenclature

The section provides descriptions of terms commonly used throughout the report to refer to various cable systems components:

Additional PPWS anchorages - the weldments that anchor the additional PPWS, found only in the side spans, at the tower tops.

Anchor Chamber - the open space into which the main cables enter, are deviated over the splay saddles and splay outwards until they reach the anchor wall.

Anchor Wall - the wall upon which the PPWS anchorages are fixed within the anchor chamber.

Main Cables - each of the two main cables is made up of two individual cables of prefabricated steel wire strands grouped and compacted into two single cables of circular cross section.

PPWS - Preformed parallel wire strands that collectively make up the main cable.

PPWS socket - the end termination of the PPWS strand

PPWS anchorage - the assembly that secures the PPWS socket to the concrete of the anchorage foundation. This is formed of the anchor bars and the crosshead slab. The anchor bars connect the PPWS to the crosshead, which is stressed down onto the anchorage wall.

The following abbreviations are used in the tables of results:

TT - Theoretical cable intersection point at Tower

TS - Theoretical cable intersection point at Splay saddle

TA - Theoretical cable intersection point at Anchorage

4 Materials

The mechanical properties of the principal cable system construction materials, relevant standards to which they are specified and partial material factors for design are described in this section. The list of materials and their specifications is not intended to be exhaustive but to provide a useful summary relevant to those elements of the cable system described in this report in particular. Further information can be found within the design and construction specifications GCG.F.05 and GCG.G.03.

4.1 Main cable

4.1.1 PPWS strands

The main cable shall be formed of PPWS strands, each of which is initially fabricated with a fixed number of wires in a regular hexagonal formation. The cold-drawn steel wire used to fabricate the main cables will be in accordance with UNI EN 10264 Parts 1 and 2, Class A, hot-dip galvanised wire, as supplemented and revised in the construction specification, GCG.G.03.03 Section 3. The minimum selected wire characteristics are summarised below:

Parameter	Value
Ultimate tensile strength (f_u)	>1860 MPa
Yield (f_y)	>1350 MPa
Young's' Modulus (E)	>200 GPa ± 5%
Elongation at failure	4 % (250 mm gauge length)
Wire diameter (Φ)	5.40 mm (average)
Galvanising	>300 g/m^2

Table 4-1: Characteristics of the main cable wire

PPWS strands will be fabricated in accordance with EN 12385 with sockets complying to EN 13411. Sockets will be detailed so as to achieve the full breaking capacity of the strand.

The design of the main cable will be carried out in accordance with Table 7 of the Design Basis, where the following partial material factors are specified on the ultimate tensile strength.

Partial material factor	Value
SILS axial resistance	1.40 (uniform stress across cable section)
ULS axial resistance	1.67 (uniform stress across cable section)
SLS axial resistance	2.10 (uniform stress across cable section)

Table 4-2: Partial material factors for the main cable tension

The partial material factor at SILS is not specified in GCG.F.04.01 and the above is proposed. The partial material factors on fatigue for load-bearing elements in bridges carrying railway are defined in RFI Instruzione 44F where:

Partial material factor	Value
Fatigue	1.35

Table 4-3: Partial material factors for fatigue

Where the main cable strands are required to resist loads applied through friction at their surface e.g. at cable clamps and saddles, this is assumed to occur at a clean interface between two galvanised elements. The relevant nominal coefficient of friction and partial material factors are given here: (the coefficient of friction to be adopted has been specified in the design specification GCG.F.05.03 paragraph 10.7.2).

Parameter	Value
Coefficient of friction (μ)	0.2
ULS slip	1.65
SILS slip	1.50

Table 4-4: Partial material factors for slip

The partial material factor at SILS is not specified in GCG.F.04.01 and the above is proposed. The transverse contact pressure on wires will be limited to ensure that the ultimate breaking strength of the wires is not significantly reduced. The partial factor below is that given in EN 1993-1-11.

Partial material factor	Value
ULS transverse pressure	1.00

Table 4-5: Partial material factors for transverse wire pressure

4.1.2 Wrapping wire

After compaction, the main cable shall be wrapped with steel wrapping wire. This shall be round wire formed from drawn soft-annealed steel wire in accordance with BS 1052.

Parameter	Value
Ultimate tensile strength (f_u)	570 - 730 MPa
Elongation at failure	8% (250 mm gauge length)
Wire dimensions	3.5 mm dia.
Galvanising	>300 g/m^2

Table 4-6: Characteristics of the wrapping wire

4.2 Anchorage crossheads

The anchor plates shall be cast steel as given in Table 4-7. The anchor bars, complete with nuts and washers, shall be grade 10.9 in accordance with GCG.G.03.02 and NTC 08, section 11 and listed in Table 4-7. Post-tensioned strands, complete with dead and live end anchorages and ducts are described in a separate report on the anchor block design.

Component	Material	Yield Stress MPa	Tensile Strength MPa
Crosshead	Cast steel, grade G20Mn5+QT (1.6220) to UNI EN 10340	300 (all ruling sections)	650
Anchor bars	36NiCrMo16 (grade 10.9)	900	1000

Nuts	42CrMo4 (class 10)	-	1000

Table 4-7: Design parameters of anchorage components

The material partial factors (safety coefficients) used to verify the steel elements are in accordance with NTC08 Sections 4.2.4.1.1, 4.2.4.1.4, 4.2.8.1.1, 4.2.8.2 and are listed in Table 4-8.

Verification	Partial Factor
Resistance of class 1, 2, 3 and 4 sections and castings	γ_{M0} = 1.05
Resistance to fracture of sections and plates under tension (weakened by holes), bolts in shear and tension, connected plies in bearing and welds	γ_{M2} = 1.25

Table 4-8: Material partial factors for steel elements used in the anchorages

5 Design Principles

Brief summaries of the relevant principles are given alongside the verifications below. They are further described in the Specialist Technical Design Report.

6 Main cable, Design and Verifications

6.1 Design description

The main cable will be erected by the Preformed Parallel Wire Strand (PPWS) method, using a hexagonal-staggered arrangement of strands aligned to provide vertical 'columns' of strands. The majority of strands run for the full length of the main cable, from anchorage to anchorage. A small number of additional PPWS strands are provided in the Sicily and Calabria side spans.

The strand arrangement and typical main cable section is shown in Figure 6-1. A summary of the composition of the main cable is given in Table 6-1. Further details of the main cable construction, geometry and materials can be found in the relevant design drawings.

The arrangement with vertical columns has been selected to optimise constructability and is as close as possible to a 21-column regular hexagonal arrangement. The details of the arrangement have been carefully developed to ensure zero strand crossings and to minimise strand twisting, whilst being fully consistent and compliant with the requirements of the looped post-tensioned (PT) anchorage strands. Each PPWS strand is anchored by a single, looped PT tendon aligned directly beneath it at the anchor wall. In almost all cases, the strand arrangement has been optimised such that each looped PT tendon anchors two PPWS strands, one at each of its ends.

Figure 6-2 and Figure 6-3 indicate how the strand arrangement is organised at the tower saddles and splay saddles to minimise wire crossing.

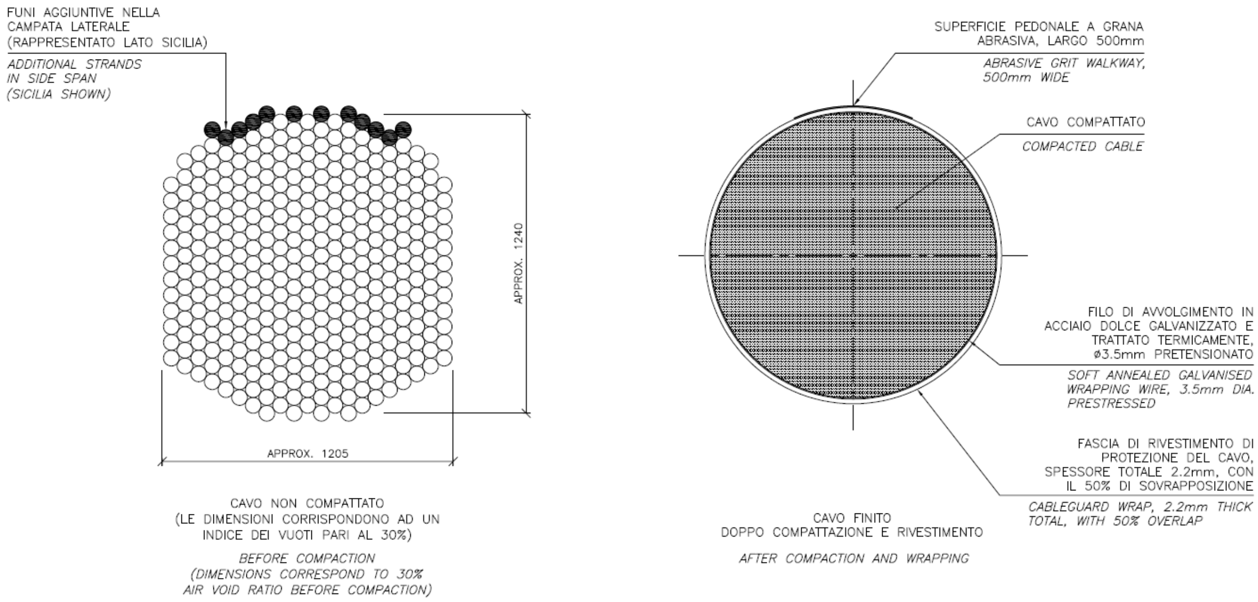

Figure 6-1: Main cable strand arrangement

DIMENSIONI E GEOMETRIA DEL CAVO PRINCIPALE
MAIN CABLE DIMENSIONS AND GEOMETRY

	CAMPATA LATERALE SICILIA SICILIA SIDE SPAN	CAMPATA PRINCIPALE MAIN SPAN	CAMPATA LATERALE CALABRIA CALABRIA SIDE SPAN
N. DI FUNI *NO. OF STRANDS*	361	349	357
N. DI FILI PER FUNE *NO. OF WIRES PER STRAND*	127	127	127
DIAMETRO FILO (mm) *WIRE DIAMETER (mm)*	5.40	5.40	5.40
N. TOTALE FILI PER 1 DEI 4 CAVI *NUMBER OF WIRES/CABLE (1 OF 4 CABLES)*	45847	44323	45339
AREA DI CIASCUNO DEI QUATTRO CAVI (m^2) *CABLE CROSS-SECTIONAL AREA (1 OF 4 CABLES) (m^2)*	1.050	1.015	1.038
DIAMETRO DI CIASCUN CAVO ESCLUSO RIVESTIMENTO (m^2) *COMPACTED CABLE DIAMETER EXCL. WRAPPING (m)*	1.285	1.263	1.278

1) DIAMETRI CORRISPONDONO AD UNA PERCENTUALE DEI VUOTI PARI AL 19 % DOPO LA COMPATTAZIONE
1) DIAMETERS CORRESPOND TO 19% AIR VOID RATIO AFTER COMPACTION

Table 6-1: Main cable dimensions and geometry

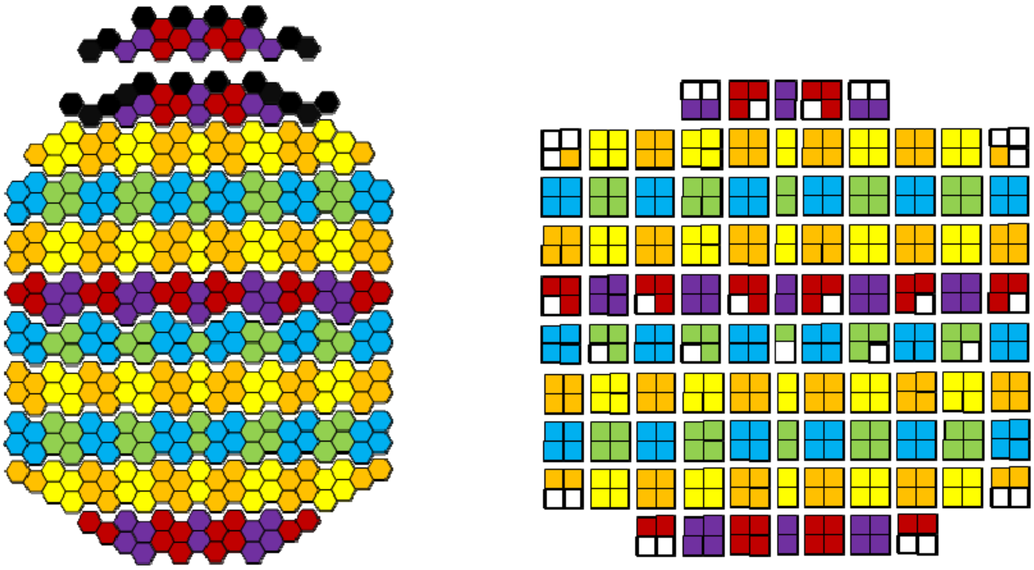

Figure 6-2: Main cable strand arrangement in suspension (left) and at tower saddle trough (right), only one main cable shown (the illustration is idealised). Calabria is shown above Silicia for the top trough plate.

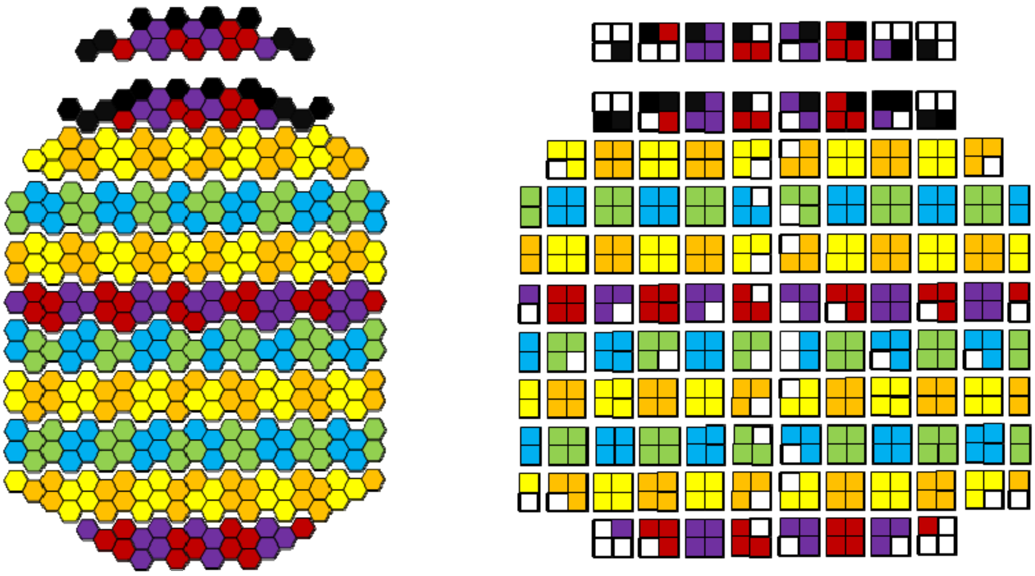

Figure 6-3: Main cable strand arrangement in suspension (left) and at splay saddle trough (right), only one main cable shown (the illustration is idealised). Calabria is shown above Sicilia for the top trough plate.

6.1.1 Strand arrangement

The vertical staggered hexagonal strand arrangement has been optimised using the following principles:

- Individual PPWS strands remain in the same column within the hexagonal arrangement of strands throughout their length from one anchorage to the other to minimise the risk of strand crossing.
- There are ten trough plates at both the tower and splay saddles. PPWS strands are located within the same trough plate at both positions. The number of trough plates is minimised.
- All additional strands in each side span (shaded black in Figure 6-2 and Figure 6-3) are located within the uppermost trough plate at the splay saddle to ensure they can be placed after the rest of the main cable is erected.
- The arrangement at the tower saddle is as compact as possible to minimise main cable spacing at 2.000 m. This compact arrangement results in 21 columns of strands in the tower saddle.
- The arrangement of the crossheads (and hence the PPWS strands) on the anchor wall is influenced to a large extent by the requirements for the PT strands used to anchor them. The PT tendons are arranged in concentric loops behind the crosshead such that each PT tendon anchors two PPWS strands, one at each of its ends. This looped arrangement dictates that an even number of strand columns is required at the anchor wall. The 21-column arrangement at the tower saddle is therefore transformed to a 22-column arrangement at the splay saddle. To avoid strands crossing one another, this additional column is created by splitting the central vertical column in two (see Figure 6-3). This transformation results in a number of 'empty' positions where some PT tendons only anchor one PPWS strand, the other end being stressed down to a crosshead that does not carry its full complement of strands (Figure 6-4). However, the arrangement has been chosen to minimise this wherever possible.
- The compact dimensions of the anchor wall means the arrangement of PT tendons needs to be optimised to ensure the minimum spacing between PT anchorages is acceptable. To achieve acceptable minimum spacings, the PT tendons are arranged in a regular rectilinear grid at the anchor wall with constant spacing of 600 mm vertically and 422.5 mm horizontally. To ensure an efficient crosshead design the arrangement of PPWS strands at the anchor wall is also set out on this rectilinear grid.
- The arrangement of strands within the splay saddle trough plates is intended to mirror the

arrangement of anchorage crossheads on the anchor wall exactly. Each crosshead anchors one group of up to 4 PPWS strands which occupy the same slot in the splay saddle trough. This is done to simplify erection and to avoid the need to cross or twist the strand. By reflecting the arrangement in the splay saddle at the anchor wall exactly, the small angular differences between the theoretical setting out lines used to align the crossheads (and PT tendons behind) and the actual alignment of the individual strands are minimised.

The above considerations results in the strand arrangements indicated in Figure 6-2 and Figure 6-3 and on the relevant design drawings.

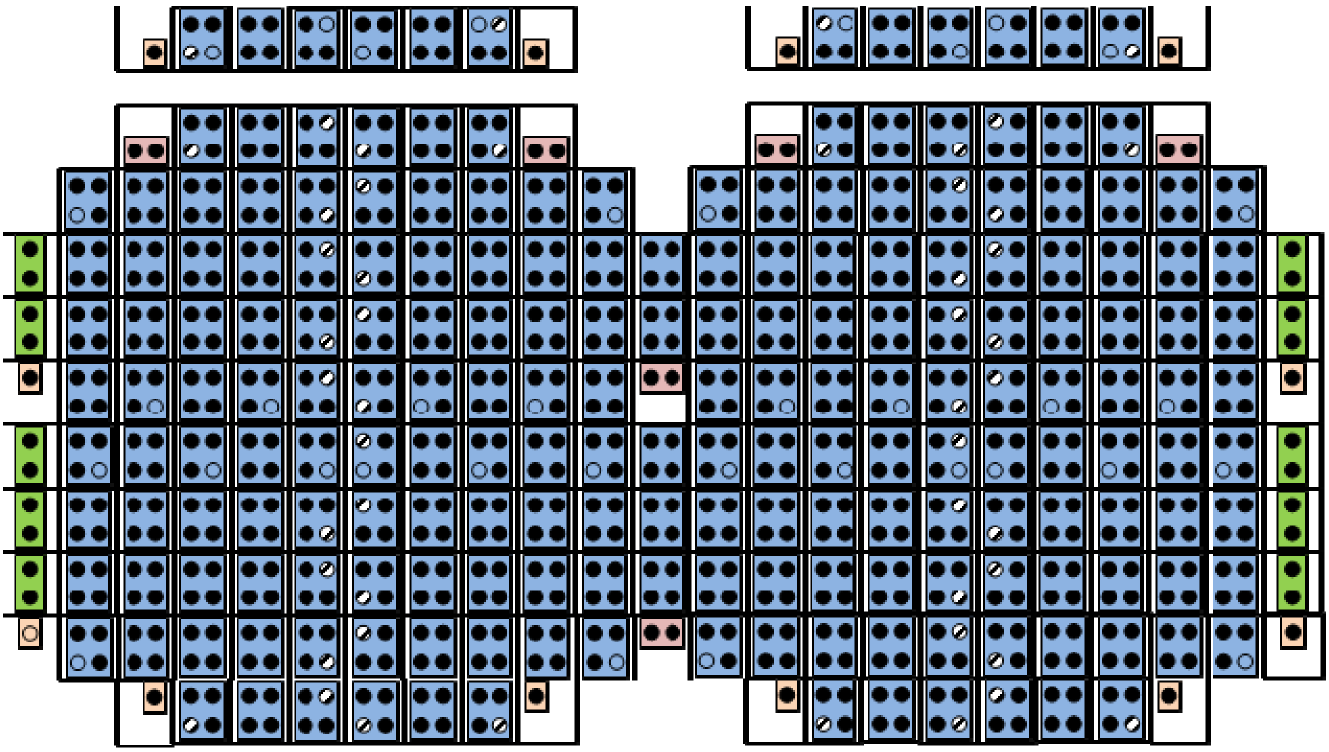

Figure 6-4: Main cable strand arrangement at the anchorage crossheads, both main cables shown. Calabria is shown above Silicia for the top row.

6.2 Design principles

The main cable is the primary load bearing element for the cable system. Under the Design Basis document the following assumptions are made:

- The main cable is a 'primary' component, and as such is critical and non-replaceable.
- Its design life is equal to that of the bridge, i.e. 200 years.
- It must survive under SILS loading combinations with only Repairable Damage (RD).
- It is not subject to local QL load combinations, only QA/QR.

6.3 Axial force

The main cable has a capacity equal to the cross-sectional area times the characteristic ultimate strength of the wire reduced by the partial material factor given at the relevant limit state. The design axial capacity is therefore:

$$F_{Ed} \leq \frac{A \cdot f_{uk}}{\gamma_M}$$

Partial material factors are given in Table 4-2. In conjunction with the cross-sections described in Figure 6-1, this results in the following axial capacities:

Axial capacity			Sicilia	Main	Calabria
SLS	Ns	MN	930	899	920
ULS	Ns	MN	1169	1131	1157
SILS	Ns	MN	1395	1349	1380

Table 6-2: Main cable axial capacity (one of four cables)

6.3.1 Reference condition

Due to the predominance of permanent loading, the main cable carries a very significant proportion of its design tension in the reference condition. Total axial load and Utilisations given as a proportion of the SLS axial capacity are shown below.

			Sicilia			Main			Calabria		
			TA	TS	TT	TT	Mid	TT	TT	TS	TA
Reference	Ns	MN	670	672	709	691	645	691	702	671	670
	Ns	MPa	638	640	675	681	635	681	676	647	645
	Ns	UR	**0.72**	**0.72**	**0.76**	**0.77**	**0.72**	**0.77**	**0.76**	**0.73**	**0.73**

Table 6-3 - Main cable tension in Reference condition (IBDAS 3.3f, c3, p1100 e2)

The approximate breakdown of force in the reference condition is given below (at the Sicilia tower on the main span side):

Reference condition			Ns
PP	Steel - Suspended deck	%	29.3
PP	Steel - Main cable	%	54.6
PP	Steel - Hangers	%	1.1
PN	Surfacing	%	4.0
PN	Other	%	11.1
PP+PN	TOTAL	%	100

Table 6-4 - Approximate breakdown of main cable tension in Reference condition (IBDAS 3.3f, c3, p1200 e2)

6.3.2 Limit states

The main cable section size is generally governed by the SLS2 limit state. Utilisations at this and other limit states are given in Table 6-5 below:

			Sicilia			Main			Calabria		
			TA	TS	TT	TT	Mid	TT	TT	TS	TA
SLS2	min Ns	MN	498	499	534	534	512	536	538	504	501
	max Ns	MN	879	881	918	883	812	887	906	878	878
SLS2	min Ns	MPa	474	475	508	526	505	528	518	485	482
	max Ns	MPa	837	839	874	870	800	874	873	845	846
SLS2	min Ns	UR	**0.535**	**0.537**	**0.574**	**0.594**	**0.570**	**0.596**	**0.585**	**0.548**	**0.545**
	max Ns	UR	**0.945**	**0.947**	**0.987**	**0.982**	**0.903**	**0.987**	**0.985**	**0.954**	**0.955**
ULS	min Ns	MN	404	405	440	459	460	465	455	414	409
	max Ns	MN	1076	1079	1119	1064	981	1074	1102	1074	1077
ULS	min Ns	MPa	385	386	419	452	453	458	438	398	394
	max Ns	MPa	1025	1027	1065	1048	966	1058	1061	1034	1037
ULS	min Ns	UR	**0.346**	**0.347**	**0.376**	**0.406**	**0.407**	**0.411**	**0.393**	**0.358**	**0.354**
	max Ns	UR	**0.920**	**0.922**	**0.957**	**0.941**	**0.867**	**0.950**	**0.952**	**0.929**	**0.931**
SILS	min Ns	MN	370	372	419	487	507	485	451	390	386
	max Ns	MN	1076	1078	1104	999	887	1019	1071	1063	1065
SILS	min Ns	MPa	352	354	399	480	499	478	434	376	372
	max Ns	MPa	1025	1026	1051	984	874	1003	1032	1024	1026
SILS	min Ns	UR	**0.265**	**0.266**	**0.300**	**0.361**	**0.376**	**0.360**	**0.327**	**0.283**	**0.280**
	max Ns	UR	**0.771**	**0.773**	**0.791**	**0.741**	**0.658**	**0.755**	**0.777**	**0.771**	**0.772**

Table 6-5 - Main cable tension at SLS2, ULS and SILS limit states (IBDAS 3.3f, c3, p1100 e2)

SLS2 forces are highest in Combination 6, PP+PN+QA+VS+VT. This is given in IBDAS combination 6536.

ULS forces are highest in Combination 7, PP+PN+QA+VS+VT, given in IBDAS combination 6517.

SILS forces are highest in Combination 2, PP+PN+QR+VS+VT, given in IBDAS combination 6812.

Overall, the main cable section size is governed by SLS2, Combination 6, PP+PN+QA+VS+VT, taken on the main span side of the Calabria tower. A breakdown of the loading for this combination and location is given in below:

SLS2 Combination PP+PN+QA+VS+VT			IBDAS Case	Ns
PP+PN	Permanent load	MN	1	691
QA	Road load	MN	501	56
QA	Road braking / acceleration	MN	511	0
QA	Rail load	MN	551	96
QA	Rail braking / acceleration	MN	561	0
VS	SLS2 Seismic (Time History)	MN		35
VT	Temperature (Differential)	MN	4510	5
VT	Temperature (Uniform)	MN	4520	4
	TOTAL	MN		887

Table 6-6 - Approximate breakdown of governing main cable tension (IBDAS 3.3f, c3, p1100, e2)

A more general summary of variable loadings from which other combinations and locations may be derived is given in Table 6-7 below. Values are unfactored unless stated otherwise.

				Sicilia			Main			Calabria		
				TA	TS	TT	TT	Mid	TT	TT	TS	TA
PP+PN	Permanent	1	Ns	670	672	709	691	645	691	702	671	670
QA	Road load	501	min Ns	0	0	0	0	0	0	0	0	0
QA			max Ns	54	53	55	56	51	56	55	53	54
QA	Road braking	511	min Ns	0	0	0	0	0	0	0	0	0
QA			max Ns	0	0	0	0	0	0	0	0	0
QA	Rail load	551	min Ns	0	0	0	0	0	0	0	0	0
QA			max Ns	95	94	96	95	89	96	95	94	94
QA	Rail braking	561	min Ns	0	0	0	0	0	0	0	0	0
QA			max Ns	0	0	0	0	0	0	0	0	0
VV	Static wind	4000	min Ns	-4	-4	-4	-1	0	-1	-4	-4	-4
VV			max Ns	10	9	10	8	8	8	10	10	10
VT	Temperature	4510	min Ns	-5	-5	-5	-4	-4	-4	-5	-5	-5
VT	(Differential)		max Ns	5	5	5	4	4	4	4	4	5
VT	Temperature	4520	min Ns	-5	-5	-5	-5	-5	-5	-5	-5	-5
VT	(Uniform)		max Ns	5	5	5	5	5	5	5	5	5
VS	SLS2 Seismic	8000	min Ns	-52	-51	-48	-29	-20	-33	-41	-47	-48
VS	(RSM)		max Ns	52	51	48	29	20	33	41	47	48
VS	SLS2 Seismic		min Ns	-50		-49	-33	-17	-31	-39	-46	-46
VS	(Time history)		max Ns	50		48	31	18	35	44	49	49
VS	ULS Seismic	8000	min Ns	-115	-113	-105	-64	-43	-72	-91	-104	-105
VS	(RSM)		max Ns	115	113	105	64	43	72	91	104	105
VS	ULS Seismic		min Ns	-111		-109	-74	-38	-68	-88	-104	-105
VS	(Time history)		max Ns	110		105	70	40	79	97	108	109
VS	SILS Seismic	8000	min Ns	-127	-125	-116	-71	-48	-80	-100	-115	-116
VS	(RSM)		max Ns	127	125	116	71	48	80	100	115	116
VS	SILS Seismic		min Ns	-123		-120	-82	-42	-75	-98	-115	-116
VS	(Time history)		max Ns	122		117	77	45	87	108	119	121

Table 6-7 - Main cable tension at SLS2, ULS and SILS limit states (IBDAS 3.3f, c3, p1100 e2)

6.3.3 Effect of hanger rupture and replacement

The effects of hanger rupture on axial stresses in the main cable have been checked. Hanger rupture is considered as the loss of both hanger strands supporting the end of any one crossbeam.

Hanger rupture is considered as an Accidental load scenario at ULS. As such, it need only be considered to occur in combination with loading specified under the reduced accidental load combinations within the Design Basis. The table below gives an envelope of main cable tensions considering the individual rupture of hangers 1, 2, 3, 4, 5, 6, 7, 30, 45 and 60 in turn. The relevant utilisations at ULS are given.

			Sicilia			Main			Calabria		
			TA	TS	TT	TT	Mid	TT	TT	TS	TA
Rupture	min Ns	MN	680	681	716	701	654	654	701	711	695
	max Ns	MN	691	692	726	709	662	662	709	719	703
Rupture	min Ns	MPa	647	649	682	690	644	644	691	685	669
	max Ns	MPa	658	659	691	699	652	652	698	693	677
Rupture	min Ns	UR	0.58	0.58	0.61	0.62	0.58	0.58	0.62	0.62	0.60
	max Ns	UR	**0.59**	**0.59**	**0.62**	**0.63**	**0.59**	**0.59**	**0.63**	**0.62**	**0.61**

Table 6-8 - Main cable tension under ULS Hanger rupture scenario (IBDAS 3.3b, p1100 e1)

It can be seen from inspection that hanger rupture is not an onerous condition for the main cable.

The effects of hanger replacement on main cable tension have also been considered. If carried out without traffic restriction or use of a temporary hanger arrangement, replacement results in overstress in the suspended deck girder and in adjacent hangers. It is therefore proposed to adopt a temporary hanger arrangement during hanger replacement and as a result, increased load effects in the main cable will be negligible.

6.4 Secondary Stresses

In addition to the verifications carried out on primary axial stress in section 6.3, the magnitude of secondary bending stresses at the tower saddle have been estimated in order to verify that the design margin provided by partial material factors specified in Table 4-2 is sufficient to allow for the magnitude of the secondary stresses expected.

6.4.1 Background

Secondary stresses are generated by all deflections of the main cable that occur after main cable erection is complete. The cable is restrained against deflecting as a pure axial element by the saddles and cable clamps and this restraint sets up non-uniform stress distributions or secondary stresses across the main cable section.

The degree of restraint is significantly affected by the presence of pre-stressed wire wrapping, which generates inter-wire friction and prevents full-slip that would otherwise significantly reduce bending stresses. Therefore, when calculating secondary stresses there are two sets of deflections to consider:

- Deflections that occur after cable clamp erection but prior to wrapping. This phase includes application of all suspended permanent load since the cable is wrapped only after deck erection is complete. Rotations will occur at the tower saddle and also at cable clamp locations when permanent loading is applied. Inter-wire slip is free to occur away from clamps.
- Deflections that occur after wrapping. These are due to application of live loading. Rotations will occur at the tower saddle and also at cable clamp locations when live load is applied. Inter-wire slip is resisted by friction generated by wire wrapping.

These deflections are almost always highest at the tower saddles (usually in the side span if the side spans are suspended, but otherwise in the main span) and at the cable clamps closest to the tower. Secondary stresses are therefore highest in the same locations.

The final section of main cable closest to the tower is left unwrapped in order to reduce the restraint to slip and therefore to reduce secondary stresses in these locations.

In a similar fashion, it is often possible to significantly reduce the secondary stresses that occur in a main cable by erecting the cable clamps immediately adjacent to the saddle after the deck is erected and immediately before cable wrapping. This is possible since the clamp does not usually have a hanger connected to it although can be inconvenient in terms of the construction sequencing.

The saddle is detailed with a large radius in order to limit secondary stresses due to local bending as rotation occurs and the wire tangent point moves.

6.4.2 Acceptance criteria for secondary stresses

The combined stress is the linear superposition of the primary stress, f_T and the total of the secondary stresses f_B:

$$f = f_T + \sum_i f_{Bi} \leq \frac{f_u}{\gamma_{m2}}$$

Secondary stresses are those induced by bending of the main cable section that generates variations between the tensions in the individual wires across the main cable section as described above.

Although not a formal requirement under the Design Basis, it is proposed here to limit the combined axial stress in any main cable wire to yield as a preliminary acceptance criterion for the secondary stresses. The acceptance criterion for limit state checks on the combined stress is therefore to limit the axial stress in any individual wire in the main cable section to:

$$f \leq \frac{f_u}{\gamma_{m2}}$$

The equivalent partial material factor applied to the combined stress at SLS2 is given in Table 6-9 below:

Limit State	Partial factor, γ_{m2}
SLS2	1.33 (75% UTS)

Table 6-9 Partial material factors on main cable combined stress

The partial material factor at SLS is intended to limit the allowable stress to yield. Yield is taken as the 0.2% proof stress. The construction specification requires that the wire proof stress is 75% of UTS and the partial material factor specified limits the allowable stress to this.

It is not appropriate to impose a limit on the combined stress at ULS or SILS since yield or rupture of individual wires does not represent an ultimate limit state for the main cable.

6.4.3 Calculation Method

This section should be read in accordance with (Wyatt, 1960) from which the nomenclature and approximate analytical solutions presented here are generally taken and adapted. Nomenclature and values used for key parameters are given in §6.4.7.

The combined stress in the main cable can be considered as being a linear summation of the following effects:

$$f = f_T + f_{B1,2} + f_{B4} + f_{B8,9}$$

- f_T Primary axial stress
- $f_{B1,2}$ Secondary local bending stress
- f_{B4} Secondary stress generated by permanent main cable deflections, under restraint from the cable clamps
- $f_{B8,9}$ Secondary stress generated by live load main cable deflections, due to restraint from wire wrapping (and possibly from cable clamps if slip length is long)

The primary axial stress, f_T is the uniform axial tension taken directly from the global IBDAS model where the main cable is modelled with tension only elements.

The secondary local bending stress, $f_{B1,2}$ is the bending stress across the individual wires (i.e. taken over the 5.40 mm diameter) assuming full slip between wires in the main cable section. The solutions for this are well established (see (Wyatt, 1960), (CIP, 2002) and others). For a fixed-ended wire that is free to bend without restraint (e.g. at a cable clamp location), the solution is;

$$f_{B1} = 2\psi_0\sqrt{E f_T}$$

where ψ_0 is the local bending angle of the wire entering or exiting the cable clamp. The stress thus calculated may be reduced by a local radius at the clamp entry or exit. Where the wire is guided over a saddle of fixed radius, R, the local bending stress is limited and replaced by the following solution:

$$f_{B2} = E\frac{r}{R}$$

After main cable spinning, or erection of PPWS strands is complete the only stresses in the main cable are f_T and f_{B2} at the saddles. All wires are able to slip along one another so that deformations of the main cable do not generate secondary stress. The situation changes after cable clamp erection.

After the cable clamps are erected, slip between the wires forming the main cable is locally prevented at every cable clamp position. This allows a bending stress distribution to be imposed on the main cable if a cable clamp rotates. It is assumed here that all permanent suspended load is added to the main cable after cable clamp erection and before wire wrapping. This generates a significant rotation of the main cable at the tower top, φ_1. Because of the no-slip condition at the cable clamps, this results in a compatible rotation at all cable clamps. The rotation is highest for the clamp closest to the tower, decreasing rapidly such that (i) net rotation at midspan is zero, and (ii) main cable rotation at one tower generates very small rotations of clamps adjacent to the other tower.

Rotation of the clamps however, generates a significant secondary stress distribution in the panel adjacent to the towers. The clamp rotations cannot be predicted using a global analysis model where the main cable is modelled with axial only truss elements. Therefore unless a special FE element type is developed for use in a global model, further investigation requires specific FE modelling in which individual wires or groups of wires are modelled. Alternatively, approximate analytical solutions from a physical model could be used.

An example of such a solution is presented in Wyatt (1960), which should give a reasonable estimate. The secondary stresses generated by rotation of the cable clamps under application of permanent loading is represented by f_{B4} given in Wyatt (1960) equation 6.

$$f_{B4} = \left(1 - K - \frac{2}{n}\right)\frac{Ea\phi_0}{l_h}$$

The situation changes when prestressed wire wrapping is added. Secondary stresses generated by further rotation of the main cable after permanent load is applied (i.e. by live load) should therefore be calculated using a different expression.

Even after cable clamps are erected, the wires making up the main cable section are free to slip along one another away from the clamp. Wrapping the cable with stressed wrapping wire generates radial compression and inter-wire friction. The main cable is therefore able to resist a limited but varying secondary stress distribution within the length between cable clamps. The secondary stress distribution is limited by the longitudinal shear capacity generated by the inter-wire friction. Due to the 'bending stiffness' given to the main cable by the wire wrapping, secondary stresses generated by live load rotations are significant and for a given rotation, will be higher than would be predicted if the wrapping were neglected. Again, the required rotations within the main cable cannot be predicted using a global analysis model where the main cable is modelled with axial only truss elements. Either a special element type or an approximate analytical solution is required.

A solution is presented in (Wyatt, 1960). The secondary stresses generated by rotation of the main cable under live loading is represented by f_{B8} given in Wyatt (1960), equation 22.

$$f_{B8} = (\mu + \ln q)\sqrt{\frac{sE\psi_1}{j}}$$

This expression also attempts to account for the reduction in restraint that results from leaving a length of cable unwrapped adjacent to the tower. However, it does assume that the cable clamps do not generate significant restraint under imposed live loading i.e. that the restraint offered by wire wrapping is high enough to prevent any tendency for wires to slip through the cable clamps.

If the effect of leaving a length of cable unwrapped is not included, the relevant expression is:

$$f_{B7} = (1.02 + \ln q)\sqrt{\frac{sE\psi_1}{j}}$$

If rotations under live loading are particularly high; or if the main cable is large, then the length over which inter-wire slip occurs can be large. If this slip length exceeds approximately two panel lengths then the cable clamps will rotate further and generate additional restraint under live loading and the expression for f_{B8} is non-conservative. In this case, the following expression should replace that for f_{B8} above:

$$f_{B8} = (\mu + \ln q)\sqrt{\frac{sE\psi_1}{j} + \frac{t_h s}{a_s j} f_c'}$$

6.4.4 Assumptions

The following assumptions have been made in this discussion and in the calculations that follow:

- The clamping force provided at cable clamps is adequate to resist slip of the wires under the rotations imposed by and on the clamps during deformation of the main cable.
- Wire wrapping is applied to the main cable after deck erection and surfacing works are complete. An average wire wrapping tension of 1500 N is assumed. This is equivalent to

approximately 150 MPa on a 3.5 mm diameter mild steel wire. This is the same as the initial tension under which wire wrapping was applied to the Storebælt bridge.

- The initial tension applied to the wire wrapping will reduce as live loading is applied due to lateral contraction of the cable (poisons ratio effects). For the same reason, the restraint provided by the cable clamps will reduce as lateral contraction leads to slackening of the cable clamp bolts, leading to increased inter-wire slip. Both of these effects are neglected in this analysis, which is therefore conservative.
- It is assumed that all cable clamps are erected and bolts fully tightened before deck erection begins. Secondary stresses could be reduced by erecting cable clamps in parallel with deck segments; however this would be unusual and is not considered here.
- The exception to this is the first cable clamp either side of the tower, which do not support hangers. These final clamps can have a significant effect on secondary stresses. It is assumed here that these clamps are installed such that slip can occur.

6.4.5 Results and Discussion

The following section provides an estimate of the magnitude of secondary stresses at the Sicilia tower saddle and an indication of the adequacy of the main cable design to withstand the secondary stresses estimated.

6.4.5.1 Main cable rotations

The following tables give the main cable rotations at the Sicily tower saddle. Results are taken from IBDAS model 3.3f.

A positive rotation in either the main span or side span represent a downwards movement of the cable. Rotations are global for the purposes of determining global cable deformations and bending stresses, that is the coincident rotation of the tower saddle that may increase or decrease rotation relative to the saddle is not included.

The following rotations occur at the Sicily tower saddle between the free-hanging condition after cable erection is complete and the reference condition after all dead load is applied. No lateral rotation occurs.

	Main span Vertical	Side span Vertical
Free-hanging to Reference	1.35	-0.88

Table 6-10 Main cable rotations in the Reference Condition (degrees)

Limit state

The following limit state rotations are the envelope values that occur at the Sicily tower saddle due to variable loading after the reference condition is reached:

	Main span				Side span			
	Vertical		Lateral		Vertical		Lateral	
	Max	Min	Max	Min	Max	Min	Max	Min
SLS2	2.20	-1.38	0.37	-0.38	0.68	-0.79	0.02	-0.01
ULS	3.40	-1.78	0.54	-0.55	1.08	-1.21	0.02	-0.02

Table 6-11 Main cable rotations at SLS2 and ULS limit states after Reference condition attained (degrees)

The values in the table above include those occurring under seismic loading (RSM method) and dynamic wind loading.

6.4.5.2 Main cable combined stress

The stresses arising from the rotations given in section 6.4.5.1 are calculated in accordance with the method presented in section 6.4.3. Utilisation Ratios are calculated in accordance with Table 6-9.

Reference condition

The main cable rotation that occurs between the free-hanging condition and the Reference condition generates secondary stresses due to the restraint provided by the cable clamps. These stresses are given in Table 6-12 below, where f_{B2} is the local wire bending stress at the saddle, and f_{B4} is the additional secondary stress arising from cable clamp rotations.

	Vertical	Primary	Secondary		Combined	UR
	Rotation $Ø_1$, deg	f_T MPa	f_{B2} MPa	f_{B4} MPa	MPa	-
Main span	1.35	681	28	72	782	0.559
Side span	-0.88	675	28	45	749	0.535

Table 6-12 Combined stresses in the Reference Condition

The secondary stresses above are calculated assuming no-slip at cable clamp locations. The required clamping force (per cable clamp bolt) required to achieve this has been estimated to be approximately 0.5 MN, which is less than the current design preload and therefore the assumption is reasonable.

Limit state

Between the Reference condition and the SLS2 limit state further secondary stresses are generated, which are additive to the secondary stresses that exist in the Reference condition. These secondary stresses are mostly generated by restraint from the wrapping wire, which reduces inter-wire slip, f_{B7}. The component f_{B7} is then reduced to allow for the relaxed restraint provided by leaving a length of main cable unwrapped adjacent to the saddle, f_{B8}. The length over which wire-slip occurs in the wrapped cable is estimated, l_q. If this exceeds approximately two panel lengths then restraint can be generated by cable clamp rotation in addition to that provided by the wrapping wire and f_{B8} is increased to f_{B9}. A summary of these components is given in Table 6-13 below to indicate the relative importance of the different restraint mechanisms.

	Vertical	Unwrapped length	Wire slip length	Secondary		
	Rotation ψ_1, deg	l_u mm	l_q mm	f_{B7} MPa	f_{B8} MPa	f_{B9} MPa
Main span	2.20	11,500	104,013	231	113	136
Side span	-0.79	11,500	62,257	138	56	79

Table 6-13 Secondary stresses at the SLS2 limit state

The length over which main cable wires slip in order to accommodate the SLS2 rotations in the main span is estimated to be almost four panel lengths and hence f_{B9} should be used to determine the total combined stress at the SLS2 limit state.

	Vertical	Primary	Secondary			Combined	UR
	Rotation ψ_1, deg	f_T MPa	f_{B2} MPa	f_{B4} MPa	f_{B9} MPa	MPa	-
Main span	2.20	870	28	72	136	1107	0.791
Side span	-0.79	874	28	45	79	1021	0.730

Table 6-14 Combined stresses at the SLS2 limit state

It is noted that secondary stresses will also be generated by lateral main cable deflections under wind load. However, the rotations are small compared to those occurring due to vertical loading and they have not been explicitly calculated here.

6.4.6 Conclusions

An analytical method for estimating the magnitude of secondary stresses has been developed and discussed, which is adapted from a physical model presented in (Wyatt, 1960). Approximate results have been presented to indicate the magnitude of secondary stresses in the main cable wire: in the Reference condition and at the SLS2 limit state.

Maximum combined wire stresses at the SLS2 limit state are estimated to be 1107 MPa, resulting in an acceptable UR of 0.79 on yield of the high-strength wire. Therefore, the partial factors currently stated in the Design Basis allow the main cable to be designed using primary stress only.

6.4.7 Nomenclature and Secondary Stress Parameters

The following is taken from Wyatt (1960). Where they are constant (or have been assumed to be), values used in the above analysis are given:

ψ_0 Local wire bending angle e.g. due to curvature over the saddle, or free bending adjacent to cable clamps.

ψ_1	Deflection angle of the end panel for load applied after wrapping i.e. global rotation of the main cable at the tower saddle under live loading
$\varnothing_0$	$= \varnothing_1 n / n - 1$, where $\varnothing_1$ is the deflection angle in the end panel adjacent to the tower i.e. the global rotation of the main cable at the tower saddle
a	Radius of main cable (average 637 mm)
E	Young's Modulus of main cable wire (200 GPa)
f_{Bi}	Secondary stress component
f_c'	Non-dimensional stress function (see (Wyatt, 1960))
f_T	Primary stress (axial tension)
j	Ratio of net area to gross area of cable section (19% voids, j = 0.81)
K	Factor by which effect of cable clamp rotation reduces between adjacent panels, a solution is presented in (Wyatt, 1960).
l_h	Typical panel length i.e. distance between cable clamps measured along main cable (30 m)
n	Number of panels (110 main span, 32 Sicily side span)
q	$= a/2r$
r	Radius of main cable wire (2.70 mm)
R	Radius of bend e.g. saddle radius (minimum 18,990 mm vertical and 16,900 mm lateral)
s	Limiting shear stress between main cable wires generated by wire wrapping $= t_w / 4a\, d_w$ where d_w, t_w are the tension force and diameter of the wrapping wire and a coefficient of friction of 0.2 is assumed for galvanised main cable wire. (d_w = 3.5 mm, t_w = 1500 N)

6.5 Fatigue

Fatigue of the main cable wire has been investigated at two locations. The first is at the PPWS anchorage socket. Here the uniform primary stress has been used since a detail classification is available in EN 1993-1-11 which along with appropriate qualification testing, accounts for secondary stresses and stress concentrations at the socket. The second location considered is the tower saddle. No detail classification exists within international codes of practice for this detail and therefore a detail classification has been proposed which will be verified through appropriate testing at Esecutivo stage. Secondary stresses are estimated analytically and explicitly included in the design stress range to allow for the fact that these are not included within the assumed detail classification.

The main cable is required to have Unlimited Life. Unlimited life is verified by ensuring that factored fluctuating stress ranges remain below the constant amplitude fatigue limit, $\Delta\sigma_D$ for the following loading:

- Most onerous fatigue train in one track plus FLM2 road load on one roadway girder.

Damage accumulation can be used for the following fatigue load combinations:

- Fatigue train on one track plus FLM3 road load on one roadway girder
- Fatigue trains in both tracks plus FLM3 road load on one roadway girder
- FLM3 road load on both roadway girders

Further details may be found in the Design Basis.

6.5.1 PPWS anchorage socket

Fluctuating axial stresses in the main cable due to the passage of fatigue vehicles have been studied assuming a uniform stress distribution across the main cable. The fluctuating stresses caused by the passage of road loading are negligible (< 1MPa) and their contribution is neglected in this section. For rail loading, the Design Basis requires that 8 different fatigue trains are considered in the design mix for Damage calculations:

- Eurocode fatigue trains EC1 to EC8

The enveloped range of axial tension experienced by the main cable under the passage of these trains is summarised in Table 6-15. The values given consider one track loaded only, are unfactored and are given for the main cable closest to the track loaded.

These axial tension ranges are converted to axial stress and factored by the relevant partial material factor, γ_m = 1.35. Dynamic impact is negligible due to the very long influence line length for main cable tension and also the remoteness of the critical detail from the applied rail loading, which must first be transferred through the railway box, crossgirders, hangers, cable clamps and along the main cable. Factored stress ranges are given in Table 6-16 below.

			Sicilia			Main			Calabria		
			TA	TS	TT	TT	Mid	TT	TT	TS	TA
EC1	Range Ns	MN	3.2	3.2	3.3	3.2	3.1	3.1	3.2	3.3	3.2
EC2	Range Ns	MN	2.6	2.6	2.6	2.6	2.5	2.5	2.6	2.6	2.6
EC3	Range Ns	MN	4.6	4.6	4.7	4.6	4.4	4.4	4.6	4.7	4.6
EC4	Range Ns	MN	2.5	2.5	2.6	2.5	2.4	2.4	2.5	2.5	2.5
EC5	Range Ns	MN	10.6	10.6	10.8	10.5	10.2	10.2	10.5	10.8	10.6
EC6	Range Ns	MN	7.0	7.0	7.1	7.0	6.8	6.8	7.0	7.1	7.0
EC7	Range Ns	MN	5.1	5.1	5.2	5.1	4.9	4.9	5.1	5.2	5.1
EC8	Range Ns	MN	9.5	9.5	9.7	9.4	9.2	9.2	9.4	9.6	9.5

Table 6-15 - Enveloped range of fluctuating axial tension in main cable tension under fatigue rail loading - one track loaded (IBDAS 3.2a, p1100 e1)

			Sicilia			Main			Calabria		
			TA	TS	TT	TT	Mid	TT	TT	TS	TA
EC1	Range	MPa	4	4	4	4	4	4	4	4	4
EC2	Range	MPa	3	3	3	3	3	3	3	3	3
EC3	Range	MPa	6	6	6	6	6	6	6	6	6
EC4	Range	MPa	3	3	3	3	3	3	3	3	3
EC5	Range	MPa	14	14	14	14	14	14	14	14	14
EC6	Range	MPa	9	9	9	9	9	9	9	9	9
EC7	Range	MPa	7	7	7	7	7	7	7	7	7
EC8	Range	MPa	12	12	12	13	12	13	13	12	12

Table 6-16 - Factored, enveloped range of fluctuating axial stress in main cable tension under fatigue rail loading - one track loaded (IBDAS 3.2a, p1100 e1)

Since the fluctuating primary stress ranges are almost constant over the length of the main cable, damage will be critical at the PPWS socket due to the stress raiser that exists here as a result of the socketing geometry and procedure. A suitable detail classification is given in EN 1993-1-11 Table 9.1, Group C, where the endurance at 2 million cycles is given as 160 MPa. From this and the S-N curve given in EN 1993-1-11 Figure 9.1, the following damage thresholds can be calculated:

Reference stress	Symbol	Endurance (cycles)	Stress range (MPa)
Detail category (reference)	$\Delta\sigma_C$	2×10^6	160
Constant amplitude fatigue limit	$\Delta\sigma_D$	5×10^6	137
Cut-off limit	$\Delta\sigma_L$	1×10^8	83

Table 6-17 - Fatigue detail classification for PPWS strand (EN 1993-1-11)

The most onerous train is EC5. By inspection of Table 6-16 the peak fluctuating stress range under a single EC5 train is approximately 14 MPa, which is considerably lower than the constant amplitude fatigue limit at 137 MPa. Therefore by inspection, the main cable has unlimited life taking a uniform stress distribution across the strand area.

The most onerous combination of fatigue loading is an EC5 train in both tracks plus an FLM3 road load vehicle on one roadway girder. This results in a fluctuating stress range of approximately 28 MPa, which is considerably lower than the cut-off limit at 83 MPa. Therefore by inspection, no fatigue damage occurs under combined road/rail loading.

6.5.2 Fatigue at the tower saddle

The results presented in this section include both primary and secondary fluctuating stresses. The assumptions made and method used to derive the secondary stresses is in accordance with that presented in section 6.4.

The partial factor on fatigue strength for fatigue of individual steel wires shall be taken as, γ_m = 1.15. This partial factor on fatigue strength is taken from NTC08 (or EN1993-1-9) for the Safe Life method with Low consequence. The Safe Life method is appropriate for the main cable wire where inspection is difficult. Fatigue failure of individual wires in the main cable has a very small effect on the overall main cable strength; redistribution of load is straightforward and disproportionate collapse will not occur. A classification of Low consequence is therefore appropriate.

An appropriate detail classification, σ_C is not given in known internationally accepted design codes. The qualification test for the high strength wire requires a fatigue strength of 380 MPa at 2 million cycles. This figure should be reduced by a factor of 1.25 as a statistical reduction to account for length effects, sample size etc to give a detail classification of approximately σ_C = 300 MPa. The reduction is taken from EN1993-1-11 Appendix A §A4.1(4).

However, it is anticipated that the fatigue strength at the saddle will be reduced compared to the high strength wire. The reduction will be primarily due to fretting where inter-wire slip coincides with transverse pressure close to the wire tangent point in the saddle and will be worst for the bottom row of wires. Fretting results in additional localised fluctuations in tensile stress, damage to the surface of the wire and accelerated crack initiation which shortens fatigue life. In open air, fretting accelerates surface corrosion, which increases surface roughness, inter-wire friction and leads to further increase in local tensile stresses and wear. Whilst the use of parallel wire; the trough plates which reduce transverse pressures; and the presence of dehumidification should all reduce the significance of fretting in the Messina tower saddle, a reduction to the fatigue strength is still anticipated.

In the absence of codified guidance, applicable test data has been sought that would give a good estimate of the fatigue strength, however none has been identified that adequately represents conditions for the wire at the Messina tower saddle. Fatigue tests on high strength wire carried out by Stretto di Messina prior to Tender (Stretto di Messina, 1992) only considered the local bending stress across a single wire; tests on 7- and 14-wire strands were planned but not completed. As a result it is recommended that appropriate testing be carried out at Esecutivo stage. A suggested test set-up is indicated below for further development at Esecutivo.

6.5.2.1 Proposed test set-up at Progetto Esecutivo

To verify the effect of fretting a saddle fatigue test is recommended at Progetto Esecutivo. An illustrative test arrangement is shown in Figure 3-1 below. It is noted that this is included for information and development of the test will be carried out at Esecutivo stage.

The saddle shall have a radius equal to that of internal radius of the grooves in the lower trough plate casting; machining tolerances and surface finishes shall be identical to the permanent works. The test saddle will be representative of the central column in the lower trough plate casting i.e. it shall contain two PPWS strands placed vertically above one another.

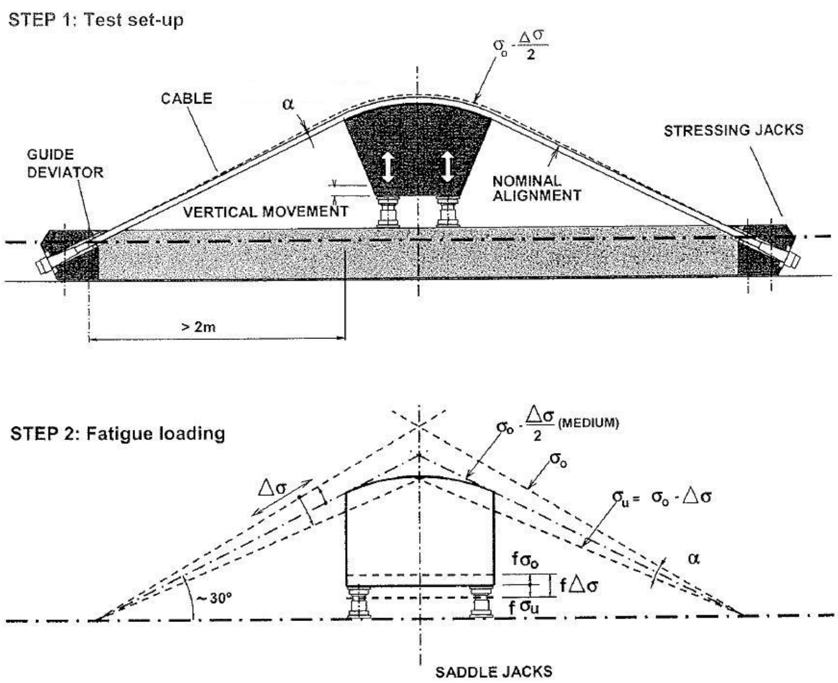

Figure 6-5 Indicative test set-up for saddle fatigue

An alternative arrangement with a single PPWS laid over a test saddle with a radius equal to half that on the tower saddle may be possible and requires further investigation. Such an arrangement would have wire pressures and inter-wire slip configurations identical to the full-scale test, although the movement of the strand tangent point at the saddle would differ.

The PPWS shall be installed and stressed to its Reference condition tension. The saddle may then be alternately raised and lowered by jacks with a frequency less than 8 Hz to induce fluctuating tension $\Delta\sigma$ in the PPWS. The test geometry shall be defined such that two conditions are met: (i) the saddle rotation, α is equal to that under the most onerous combination of fatigue loading in the full-scale; and (ii) the stress range $\Delta\sigma$ is equal to the combined stress range, Δf, calculated under the most onerous combination of fatigue loading. If the combined stress range is used then the effect of cable clamp restraint and wire wrapping need not be included in the test set-up. In order to achieve both conditions, synchronous jacking at the PPWS terminations may be necessary.

6.5.2.2 Preliminary assessment at Progetto Definitivo

Some test data is available for simple fretting tests on clamped single wires carried out in Japan and China.

Testing at Shanghai Research Institute of Materials (Qiang, Yunshu, Baoyu, & Xiangying, 1994) suggests a fatigue strength of 260 MPa at 10 million cycles for a 7 mm grade 1660 MPa wire under a transverse pressure of 180 kN/m. If the approach given in EN 1993-1-11 is assumed, this is equivalent to a constant amplitude fatigue limit of 292 MPa, which reduced by a factor of 1.25 gives 233 MPa. This is only slightly lower than that calculated from the wire qualification test (see below).

Similar tests in Japan have demonstrated that there is negligible reduction in fatigue strength for wire under a transverse pressure of less than 500 kN/m. The tests were carried out in 5.12 mm grade 1770 MPa wire.

Therefore, the indication is that whilst fretting will lead to a reduction in fatigue strength and cannot be neglected, the reduction should not be large.

For preliminary assessment of fatigue life at Progetto Definitivo a detail classification of σ_C = 300 MPa at 2 million cycles will be used, which is taken directly from the wire qualification test. The value includes the statistical reduction required by EN1993-1-11 Appendix A §A4.1(4). This gives the following damage thresholds:

Reference stress	Symbol	Endurance (cycles)	Stress range (MPa)
Detail category (reference)	$\Delta\sigma_C$	2×10^6	300
Constant amplitude fatigue limit	$\Delta\sigma_D$	5×10^6	258
Cut-off limit	$\Delta\sigma_L$	1×10^8	156

Table 6-18 - Assumed detail classification for high-strength steel wire at the tower saddle

6.5.2.3 Calculation

The fluctuating combined stress range in an individual wire may also be calculated using the expressions given in section 6.4.3. The unfactored, maximum fluctuating stress range in an individual wire is:

$$\Delta f = \Delta f_T + \Delta f_{B1,2} + f_{B3}$$

Δf_T Change in primary axial stress under fatigue loading

$\Delta f_{B1,2}$ Change in secondary local bending stress. Either due to rotation of the clamp under live load, or to movement of the tangent point and bending of an increased length of wire at the saddle

Δf_{B8} Secondary stress generated by main cable deflections under fatigue loading, due to restraint from wire wrapping

6.5.2.4 Results and Discussion

The following section provides an estimate of the magnitude of the rotations and combined stress ranges in the steel wire at the Sicilia tower saddle under the passage of combined road and rail loading. The following tables give the main cable rotations at the Sicily tower saddle. Results are taken from IBDAS model 3.3f.

A positive rotation in either the main span or side span represent a downwards movement of the cable. Rotations are global for the purposes of determining global cable deformations and bending stresses, that is the coincident rotation of the tower saddle that may increase or decrease rotation relative to the saddle is not included.

Table *6-19* gives the maximum and minimum rotations that occur due to the passage of individual road and rail fatigue vehicles. In each case only one track or roadway girder is loaded and the rotation at the most onerous main cable reported. Negligible lateral rotation occurs under the passage of fatigue vehicles.

By superposition, two governing cases are identified. The first is that under which Unlimited Life should be verified, the second is the most onerous loading under a damage accumulation calculation. Rotations are given in Table 6-20.

The stresses arising from the rotations given in section 6.4.5.1 are calculated in accordance with the method presented in section 6.4.3.

	Main span Vertical	Side span Vertical

	Max	Min	Max	Min
EC1	0.11	-0.03	0.05	-0.01
EC2	0.09	-0.03	0.03	-0.01
EC3	0.15	-0.05	0.05	-0.01
EC4	0.09	-0.03	0.04	-0.01
EC5	0.36	-0.11	0.16	-0.03
EC6	0.23	-0.07	0.08	-0.02
EC7	0.18	-0.05	0.10	-0.02
EC8	0.18	-0.05	0.09	-0.02
FLM2	0.01	0.00	0.01	0.00
FLM3	0.01	0.00	0.01	0.00

Table *6-19* Main cable rotations under passage of fatigue vehicles (degrees)

	Main span Vertical		Side span Vertical	
	Max	Min	Max	Min
EC5 + FLM2	0.37	-0.11	0.16	-0.03
2 x EC5 + FLM3	0.69	-0.20	0.30	-0.06

Table 6-20 Main cable rotations for relevant fatigue load combinations (degrees)

The range of main cable rotation at the tower saddle under the passage of fatigue trains generates a fluctuating secondary stress range in addition to the primary stress range Δf_T. The secondary stress range consists of two components: an increase in local wire bending generated as the saddle tangent point moves under cable rotation, f_{B2}, and cable bending generated by the wire wrapping restraint, Δf_{B8}. The first is the same regardless of loading; the latter is calculated in Table 6-21 below. Values are given for main span since secondary stresses have been shown to be worse here.

Main span	Vertical rotation		Wire slip length	Secondary		
	$\psi_{1,min}$ deg	$\psi_{1,max}$ deg	$l_{q,max}$ mm	$f_{B8,min}$ MPa	$f_{B8,max}$ MPa	Δf_{B8} MPa
EC1	-0.03	0.11	12,686	-4.5	13.0	17.5
EC2	-0.03	0.09	11,338	-3.7	10.7	14.4
EC3	-0.05	0.15	15,059	-6.1	16.0	22.1
EC4	-0.03	0.09	11,132	-3.6	10.5	14.1
EC5	-0.11	0.36	22,904	-12.4	32.1	44.5
EC6	-0.07	0.23	18,612	-8.8	22.8	31.6
EC7	-0.05	0.18	15,874	-6.7	19.0	25.7
EC8	-0.05	0.18	15,869	-6.7	18.8	25.5

Table 6-21 Fluctuating fatigue stress due to wire wrapping restraint under train EC1 - EC8

It can be seen that the length over which wire slip occurs is sufficiently low that the cable clamps are unlikely to generate significant additional restraint. The total factored stress range at the saddle for each single fatigue train is given in Table 6-22 below.

It can be seen that the stress range under any single train is below the cut-off limit and therefore causes no damage. Dynamic factors have been neglected in this analysis since they will not be significant for this detail due to the very long influence line length for main cable rotation and axial stress and also due to the remoteness of the detail from the applied load.

In addition to single train passes, combined stresses under two governing cases have been estimated. The first is that under which Unlimited Life should be verified (EC5+FLM2), the second is the most onerous loading under a damage accumulation calculation (2 x EC5+FLM3). Although part of the combined load, the effect of road load is negligible and can be neglected since the rotations and axial stress fluctuations are very small indeed and both primary and secondary stress fluctuations will be negligible.

Main span	Primary	Secondary		Combined stress range	
	Δf_T MPa	f_{B2} MPa	Δf_{B8} MPa	Range MPa	Factored MPa

EC1	3.2	28.4	17.5	49.1	56.5
EC2	2.5	28.4	14.4	45.4	52.2
EC3	4.5	28.4	22.1	55.1	63.4
EC4	2.5	28.4	14.1	44.9	51.7
EC5	10.4	28.4	44.5	83.3	95.8
EC6	6.9	28.4	31.6	66.9	76.9
EC7	5.0	28.4	25.7	59.1	68.0
EC8	9.3	28.4	25.5	63.3	72.8

Table 6-22 Fluctuating combined fatigue stresses under single trains EC1 - EC8

Main span	Vertical rotation		Wire slip length	Secondary		
	$\psi_{1,min}$ deg	$\psi_{1,max}$ deg	l_q mm	$f_{B8,min}$ MPa	$f_{B8,max}$ MPa	Δf_{B8} MPa
EC5+FLM2	-0.11	0.37	42,802	-12.6	32.7	45.3
2 x EC5 + FLM3	-0.20	0.69	58,486	-20.7	51.3	72.0

Table 6-23 Fluctuating fatigue stress due to wire wrapping restraint for governing load combinations

Main span	Primary	Secondary		Combined stress range	
	Δf_T MPa	f_{B2} MPa	Δf_{B8} MPa	Range MPa	Factored MPa
EC5+FLM2	10.4	28.4	45.3	84.1	96.7
2 x EC5 + FLM3	20.7	28.4	72.0	121.2	139.4

Table 6-24 Fluctuating combined fatigue stresses for governing load combinations

The total factored stress range at the saddle for the two combinations is given in Table 6-24 above.

It can be seen that the factored stress range for EC5+FLM2 is below the constant amplitude fatigue limit assumed and therefore Unlimited Life is confirmed. Furthermore, the factored stress range due to the most onerous combined loading is also below the constant amplitude fatigue limit assumed and therefore no significant damage should occur at the saddle due to road and rail loading. The detail classification assumed at the tower saddle will be verified through testing at the Esecutivo stage.

6.6 Tower saddle sliding

As the main cable passes over the saddle, the horizontal component of main cable tension can vary between the side span and main span. This change in the horizontal component of tension is balanced by a shear force applied to the top of the tower which is generated by friction between the PPWS strands and the trough plate grooves. Friction is mobilised on the bottom surface of the trough plate groove due to the vertical pressure applied by the strand; additional friction is mobilised on the sides of the trough plate groove due to the complementary lateral bursting pressures generated. Due to the significant number of vertical spacers in the tower saddle trough plates, a significant reserve of friction can be mobilised on the sides of the trough plate grooves in the Messina saddle.

The governing combination for slip in the saddles is that which maximises the ratio V_z/N_s, where N_s is the axial force in the tower saddle and V_z is the coincident longitudinal shear. This is given by Combination 7, PP+PN+QA+VS+VT at ULS and by Combination 2, PP+PN+QR+VS+VT at SILS. Assumed friction parameters are given in Table 4-4.

The Utilisation Ratios given in Table 6-25 above demonstrate the acceptability of the design, but are conservative since they do not consider the significant friction reserve available from lateral bursting pressures on the sides of the trough plate grooves.

			Vz	Ns	Vz/Ns	μ/γ_M	UR
ULS	PP+PN+QA+VS+VT (Uniform)	7517	130.7	1116			
ULS	VT	4510	1.0	4			

		Total	131.7	1120	0.118	0.121	**0.97**
SILS	PP+PN+QA+VS+VT (Uniform)		137.1	1137			
SILS	VT	4510	1.0	4			
		Total	138.1	1141	0.121	0.133	**0.91**

Table 6-25 - Maximum friction demand in the tower saddles (IBDAS 3.3d, g2EPSe1sl2s25p12)

6.7 Wire pressures

Saddle and trough plate radii are set to ensure that bending stresses and contact pressures on the PPWS wires are acceptable and do not reduce the wire strength beyond that allowed for within the partial material factors adopted.

Wire pressures are given here as kN/m and are calculated using the maximum ULS or SILS main cable tension given in Table 6-5 divided by the number of strands in the cable (see Table 6-1) and multiplied by the number of strands in each groove.

	Saddle	Main cable, Ns MN	No. strands	Strands /groove	Tension/groove MN
ULS	Sicilia tower saddle	1065	349	4	12.2
ULS	Calabria tower saddle	1078	349	4	12.4
ULS	Sicilia splay saddle	1093	361	4	12.1
ULS	Calabria splay saddle	1081	357	4	12.1
SILS	Sicilia tower saddle	1015	349	4	11.6
SILS	Calabria tower saddle	1030	349	4	11.8
SILS	Sicilia splay saddle	1108	361	4	12.3
SILS	Calabria splay saddle	1064	357	4	11.9

Table 6-26 - Main cable tension in saddle trough plate grooves

The rectangular PPWS section in each groove is approximately 24 wires wide and 22 wires deep (assuming four strands per groove). Wire side pressures are highest in the bottom trough plate in both the tower saddle and the splay saddle since the radius of curvature is lowest here. The splay saddle deviates the wire through a radius both horizontally and vertically and so a vertical (V) and horizontal (H) pressure is given.

	Saddle		Tension/groove MN	Radius (Plate A) m	Wires at groove base/side	Wire side pressure kN/m
ULS	Sicilia tower saddle	V	12.2	18.990	24	27
ULS	Calabria tower saddle	V	12.4	19.539	24	26
ULS	Sicilia splay saddle	V	12.1	5.001	22	110
		H	12.1	11.508	22	48
ULS	Calabria splay saddle	V	12.1	5.001	22	110
		H	12.1	11.615	22	47
SILS	Sicilia tower saddle	V	11.6	18.990	24	25
SILS	Calabria tower saddle	V	11.8	19.539	24	25
SILS	Sicilia splay saddle	V	12.3	5.001	22	112
		H	12.3	11.508	22	49
SILS	Calabria splay saddle	V	11.9	5.001	22	108
		H	11.9	11.615	22	47

Table 6-27 - Wire side pressures in saddles (V, vertical, H, horizontal)

The horizontal side pressures quoted at the splay saddle are increased by the lateral bursting pressures generated, but will remain within acceptable limits.

6.8 Fabrication and erection tolerances

Fabrication and construction tolerances can result in non-uniform stress distributions (i) across individual PPWS strands; (ii) across a compacted main cable section; (iii) between two main cables in a pair. These tolerances and non-uniform stress distributions are dealt with in the following manner:

1. Fabrication of the 127-wire PPWS strands will be to a construction specification requiring limits on the variation of individual wire lengths. This will require that the tension carried by any single wire will only vary by 1% compared to any other in the strand. Such a tolerance is already known to be achieved by some PPWS manufacturers and will enable the non-uniform stress distribution that results to be dealt with through the partial material factors in

design.

2. Non-uniform stress distribution across a main cable section can arise due to bending moments imposed due to main cable deformations and rotations at saddles; rotation of cable clamps under live loading; and differential temperature through the main cable section. Saddles and cable clamps are detailed in accordance with best practice to ensure that secondary stresses are minimised at these locations. The combined effect of these measures ensures than any residual non-uniformity of stress across the section can be dealt with through the partial material factors in design.
3. Non-uniform stress distribution between the two main cables in a pair due to differential heating is dealt with through global modelling. By modelling each of the four main cables independently in the global IBDAS model and applying the temperature difference as a loading, a uniform stress for each of the cable sections can be derived independently. Axial tensions given elsewhere in this report include this effect within the VT component.
4. Non-uniform stress distribution between the two main cables in a pair due to erection tolerances has been dealt with through parametric studies using simplified finite element models that are further described below.

With respect to item (3) and (4) above, the following has been assumed:

Parameter	Value
Maximum difference in effective temperature (additional VT effect)	± 1 °C
Maximum vertical tolerance on cable profile after compaction, prior to deck erection	± 75 mm (150 mm total)

Table 6-28: Parameters for determining stress difference between two main cables in a pair

The ±1°C figure has been taken from FE thermal flow simulations carried out on the closely spaced main cables during the pre-tender stage.

An erection tolerance of +/-75 mm has been assumed giving a maximum difference in level of 150 mm between the two main cables in a pair, which can reasonably be expected to be better than the tolerance on the absolute position of either cable. Although here are few reference cases of main cables erected side-by-side, the tolerance of +/-75 mm was selected based on construction experience on Japanese suspension bridges as well as Storebaelt, Second Bosporus and Irtysh (Kazakhstan). Comparison can be made with the final difference in sag between the two main cables supporting either side of the deck after cable erection is completed. This was 45 mm for Akashi and a maximum of 87 mm on other known reference projects, which compares very favourably with the conservative value of 150 mm assumed here.

6.8.1 Effect of erection tolerance

The following section addresses the issue of main cable erection tolerance and the effect this has on reference main cable profile and tension. The parametric study has been carried out using theoretical cable catenary calculations and then verified through simplified finite element modelling.

It is noted that input parameters and results from the parametric studies are approximate and are intended to illustrate the nature of the structural response and supplement the global IBDAS analysis.

6.8.1.1 Assumptions

The following assumptions are made for the purposes of the parametric study:

- The parametric study has assumed an 'initial' vertical difference in profile of 150 mm at mid span under the free-hanging cable condition prior to erection of hangers and deck units etc.
- The average sag of the two main cables in a pair under the nominal permanent suspended load is that given for the reference condition.
- The two main cables forming a pair and connected by cable clamps, have been modelled as simple, independent catenaries subject to a uniform distributed load. Only the main span has been modelled, with the tower tops considered fixed.
- The proportion of the suspended load to be taken by each of the main cables in the 'final' condition has been calculated statically from the cable clamp geometry and by iterating the catenary calculation to determine the 'final' difference in vertical profile between the two main cables. This iteration has attempted to model the 'restoring moment' applied by the cable clamps, which increases load on the 'higher' of the two main cables and reduces load on the 'lower'. This results in an increase in sag of the higher cable and a reduction in the difference in vertical profile. This situation is complicated by the difference in stiffness of the two main cables resulting from their different unstrained lengths.

- Load from hangers has been applied as a uniform distributed load onto each catenary. It is noted that the difference in level of the two main cables is a maximum at midspan and zero at the pylons and therefore, the suspended load will be distributed evenly between the two cables at the pylon, with the distribution becoming less and less even towards midspan. The loading is therefore not uniform; for the purposes of this however, an 'equivalent' average UDL has been applied over the cable length.

6.8.1.2 Results

The following results are obtained using the above assumptions. For an 'initial' vertical difference in profile of 150 mm in the free-hanging condition, the difference in main cable profile and tension at the pylon for the 'final' reference condition are given in Table 6-29 below. The table above indicates the 'final' iterated main cable profiles for a variety of hanger pin offsets This offset is directly proportional to the restorative effect of the cable clamp geometry on main cable profile.

	Free-hanging condition		Reference condition	
Hanger pin perpendicular offset from cable centreline mm	Difference in cable profile at midspan mm	% difference in cable tension at Pylon %	Difference in cable profile at midspan mm	% difference in cable tension at pylon %
0	150	0.1	142 *(144)*	0.0
600	150	0.1	24	0.5
1200	150	0.1	13 *(21)*	0.6
2000	150	0.1	8	0.6

Table 6-29 - Main cable condition for initial vertical difference of 150 mm. Numbers in parentheses are from a FE model used to verify the catenary spreadsheet calculation.

Key results from the catenary analysis have been verified using a large-displacement FE analysis. This models the mainspan of a main cable pair using simple beam elements assuming fixity of the tower tops. The cables are cross-linked by rigid slave members representing the cable clamps and perpendicular hanger lugs. Once the free-hanging condition had been determined, permanent hanger forces taken from the global IBDAS model have then been applied to the hanger lugs. It can be seen that the values offer good agreement to the approximate catenary calculations.

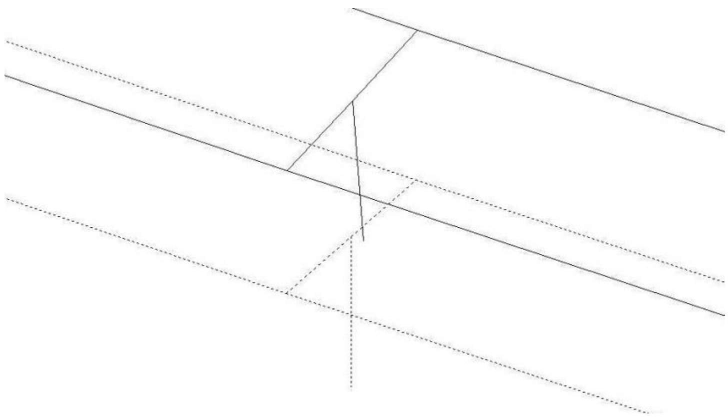

Figure 6-6 - Plot of simplified FE model showing arrangement at cable clamp

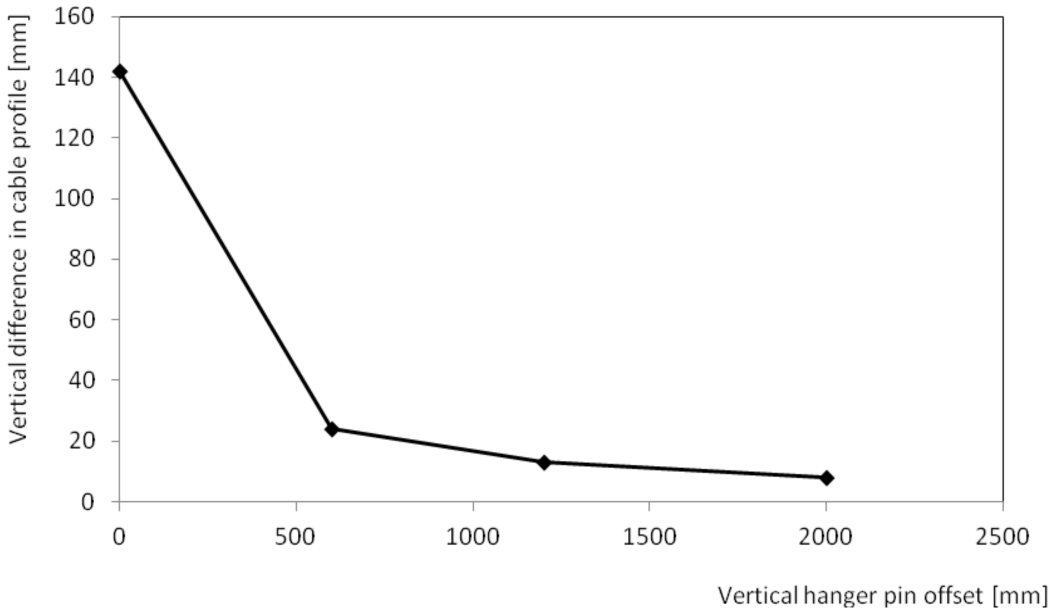

Figure 6-7 - Plot of vertical difference in cable profile against hanger pin offset at the reference condition for an initial erection tolerance of 150 mm

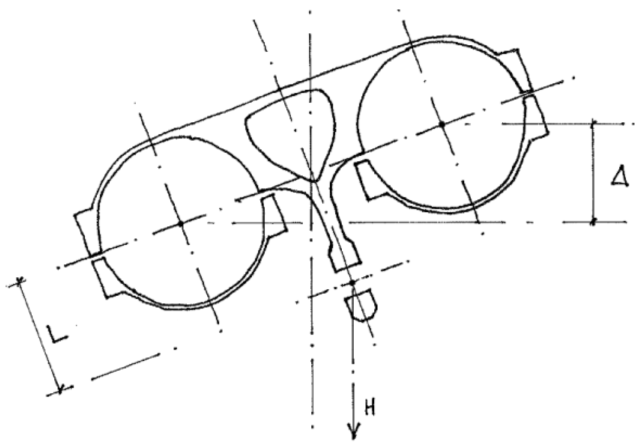

Figure 6-8 - Geometry of clamp with difference in main cable profile, Δ and hanger pin offset L.

A second study has been carried out using a constant hanger pin offset of 1200 mm for a varying initial difference in cable profile to establish sensitivity to the assumed figure of 150 mm. The results are given in Table 6-30 below.

	Free-hanging condition		Reference condition	
Hanger pin perpendicular offset from cable centreline mm	Difference in cable profile at midspan mm	% difference in cable tension at Pylon %	Difference in cable profile at midspan mm	% difference in cable tension at pylon %
1200	100	0.1	9	0.4
1200	150	0.1	13	0.6
1200	200	0.1	17	0.7

Table 6-30 - Main cable condition for a varying initial vertical difference.

6.8.1.3 Conclusions

The following conclusions are drawn from the parametric study into main cable erection tolerance:

- A cable clamp with the hanger lug perpendicular to, and below the transverse axis connecting the cable centres provides a restoring force that reduces any initial difference in vertical level. For small hanger pin offsets, the final difference in cable profile is very sensitive to this offset (see Figure 6-7).

- This sensitivity drops rapidly such that an offset greater that 500 mm offers an ever-reducing benefit. A zero-difference in cable profile is not theoretically possible.
- For the final PD solution with an offset of approximately 800 mm and a cylindrical bearing at the upper hanger pin, the total effective offset is increased by the length of the hanger socket since transverse rotation is not permitted at the hanger pin and bending will tend to occur at the neck of the hanger socket. The offset in the final design is therefore approximately 1,800 mm. By reference to Table 6-29 and for an initial erection tolerance of 150 mm this will result in a final difference in profile of approximately 15 mm in the reference condition.
- The difference in tension at reference condition, between two cables in a pair is less than 1% for the values of initial difference in cable profile considered.
- Both the initial erection tolerance of 150 mm and the final difference in profile of 15 mm need to be considered in the design of the cable clamp since they will introduce additional bending stresses.

6.8.2 Effect of temperature difference between main cables in a pair

The following section addresses the issue of a temperature difference between the two main cables forming a pair and the effect this has on reference main cable profile and tension. The parametric study has been carried out using theoretical cable catenary calculations and then verified through simplified finite element modelling.

In all cases, the reference condition has been established assuming an initial erection tolerance of 150mm. A negative (cooling) temperature load has then been applied to the higher (shorter) cable so as to exaggerate the existing profile difference. The results are shown in Table 6-31 below.

For the differential temperature loading of ± 1 °C specified in the Design Basis, the effect on main cable profile and tension is small. The effect on main cable tension is accounted for within the global IBDAS analysis. The temperature difference results in a further difference in vertical cable profile of approximately 5 mm to be accounted for in the design of the cable clamps. This is to be added to the 15 mm experienced due to the initial erection tolerance to give a total of approximately 20 mm.

	Free-hanging condition	Reference Condition	Differential temperature		
Temperature difference between cables	Difference in cable profile at midspan	Difference in cable profile at midspan	Difference in cable profile at midspan	Change in cable tension at pylon from reference condition %	
ºC	mm	mm	mm	Higher	Lower
0	150	13	13 (21)	0	0
10	150	13	87 (73)	1.6 (1.4)	-1.4 (-1.1)
20	150	13	158 (125)	3.3 (2.7)	-2.9 (-2.3)

Table 6-31 - Main cable condition under differential thermal load for a hanger pin offset of 1200 mm. Numbers in parentheses are from a simplified FE model used to verify the catenary spreadsheet calculation.

For the design of the cable clamps, a fixed tolerance of 150 mm will be used for all loading applied up to the reference condition. This is conservative since the behaviour is geometrically non-linear and the difference in level will drop from 150 mm as reference loading is applied such that when full permanent load is applied the difference in level will be substantially less.

Beyond the reference condition, a fixed tolerance of 30 mm will be used for all load applied up to hanger rupture. The value of 30 mm is conservative compared to the 15 mm calculated above. However, this is appropriate, since hanger rupture is a localised loading with respect to the main cable. By contrast, the self-correcting geometry of the cable clamp will only be effective for loads applied uniformly along the main cable and so fluctuations in load beyond the reference condition are unlikely to lead to further reductions in the level difference between the two main cables.

6.9 Aerodynamic stability

Wind tunnel testing has been carried out during Progetto Definitivo to verify the aeroelastic stability and response of the main cables. This has identified a potential risk of galloping of the main cable pair in the side spans where it is not stabilised by hangers. A concept for mitigating the side-span instability has been developed and consists of a perforated plate installed between the two main cables. This is intended to reduce the air flow that is otherwise funnelled between the two main cables and could trigger the instability mechanism. The plate could be retro-fitted if the phenomenon is observed in-service and the detailed design will ensure the plate can be easily fitted and fixed in short lengths without damaging the Cableguard wrap and keeping disruption to O&M procedures to a minimum.

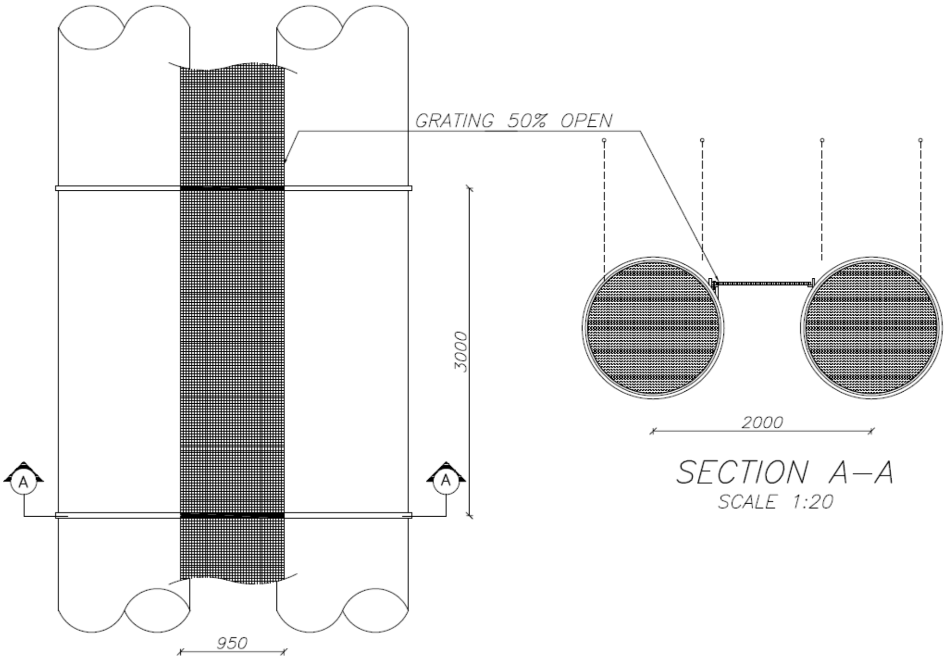

Figure 6-9 - Concept sketch for perforated plate to mitigate aerodynamic instability of the main cable

Further testing is planned during or prior to Esecutivo. This testing is intended to further investigate the phenomenon to establish whether or not the risk is acceptable and to demonstrate the feasibility of the proposed concept.

6.10 References

The following references are referred to in section 6 above:

CIP. (2002). *Cable stays - Recommendations of French interministerial commission on Prestressing.* SETRA.

Qiang, G., Yunshu, D., Baoyu, Z., & Xiangying, Z. (1994). Fretting fatigue of steel wire for bridge cable. *International Symposium on Cable-Stayed Bridges*, (pp. 351-358). Shanghai.

Stretto di Messina. (1992). *L'Opera di Attraversamento - Sistema di Sospensione - Indagini Sperimentali.*

Wyatt, T. (1960). Secondary Stresses in Parallel Wire Suspension Bridges. *Journal of Structural Division, Proc. ASCE , 86* (ST7), 37-59.

7 Anchorage crossheads and fixings, Design and Verifications

The anchorage consists of the following structural elements:

- PPWS sockets
- Anchor bars
- Anchorage crossheads

Design verifications for the PT tendons used to anchor the crossheads and checks on the bearing and bursting stresses generated within the concrete face are covered elsewhere.

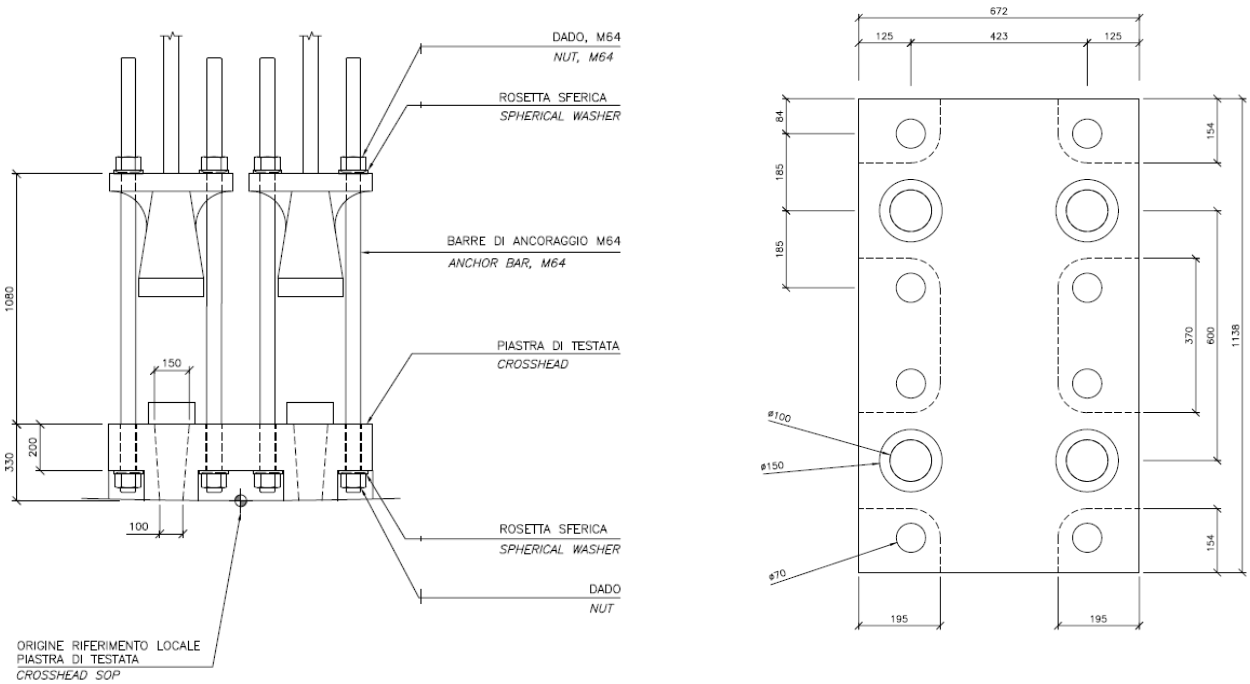

Figure 7-1: Typical anchorage crosshead arrangement

7.1 Design Description

The anchorage crosshead transfers the tension in the main cable PPWS into the concrete of the anchorage foundation. Under the Design Basis, the following assumptions are made:

- The anchorage crosshead is part of the primary suspension system and is therefore a critical, non-replaceable, 'primary' component.
- The design life of the anchorage crossheads and associated components is 200 years.
- Anchorage crossheads are required to resist SILS loading scenarios.
- Anchorage crossheads are subject to QA/QR load combinations at all limit states. They are not required to resist QL loading.

Anchorage crossheads are allowed to undergo Minimal Damage (MD) up to ULS and Repairable Damage (RD) up to SILS. Localised plasticity is therefore permissible up to ULS provided it does not affect performance of the crosshead.

7.1.1.1 Tolerances

The crosshead fulfils a secondary function of accommodating tolerances on the length and setting out of the PPWS strands. These are dealt with as follows:

1. Tolerance on the length of the PPWS strand is accommodated by the threaded length of the anchorage bars. These are designed to accommodate a tolerance of 1:15,000 on the length of the PPWS, assuming that any out of tolerance is divided equally between the two ends of the PPWS. Assuming a stressed length of approximately 5,285 m, this gives a tolerance on the final PPWS socket position of approximately ± 175 mm. The arrangement has been checked to ensure that PPWS sockets do not clash above the crosshead at this tolerance.
2. The crossheads are each set out to SOP's on the anchor wall. These SOP's also provide the alignment for the PT tendons behind. By virtue of their splay, there is a small difference between the angular alignment of the PT tendons/crosshead and the PPWS strands. This difference varies depending on the position and type of crosshead. It is accommodated by placing the M64 anchor bars in oversized holes and terminating them with a spherical seat and nut.

Where the arrangement of PPWS strands at a crosshead is not doubly-symmetric about the SOP, the break-angle between PPWS strand and PT tendon results in small resultant forces parallel to the anchor wall. These are resisted through friction generated in the bearing interface between the crosshead and anchor wall. Since the PT tendons are sized to resist the full breaking load of the PPWS strand and the partial material factor on main cable tension is relatively high, the friction capacity of the crosshead/anchor wall interface is large compared to the demand at ULS.

7.2 Design criteria

In principle, the anchorages have been designed to resist the full breaking load of the PPWS strands without rupture and with only localised plasticity that does not affect the overall structural performance or integrity of the elements involved. In general therefore, the design is robust and limit state utilisations at SLS, ULS and SILS are not governing.

7.3 PPWS Sockets

The PPWS sockets are a proprietary item to be designed by the chosen supplier of the PPWS strands. The specification will require that these are to have a capacity exceeding that of the PPWS strand itself and therefore no further design checks are required.

7.4 Anchor bars

7.4.1 Strength

The anchor bars connecting the PPWS strand to the crosshead are M64 threaded rods of resistance class 10.9 steel to UNI EN 20898-1. The rupture capacity of a pair of anchor bars is therefore:

$$F_{t,Rd} = \frac{0.9 f_{ub} A_s}{\gamma_M}$$

The PPWS tension is taken as the maximum allowable under the relevant limit state. This results in the following limit state tension capacities:

Limit state	γ_M	PPWS tension (UR=1.00) MN	Anchor bar capacity (pair) MN	UR
ULS	1.25	3.24	4.63	0.70
SILS	1.00	3.86	5.79	0.67
Rupture	1.00	5.41	5.79	0.93

Table 7-1 - Anchor bar strength utilisation

7.4.2 Fatigue

From the fluctuating axial tensions given for the main cable in Table 6-15 above, the fluctuating stress range in an anchor bar can be calculated. The fluctuating stresses caused by the passage of road loading are negligible (< 1MPa) and their contribution is neglected in this section. For rail loading, the Design Basis requires that 8 different fatigue trains are considered in the design mix for Damage calculations:

- Eurocode fatigue trains EC1 to EC8

Unlimited life for the PPWS anchor bars is verified by ensuring that factored fluctuating stress ranges remain below the constant amplitude fatigue limit, $\Delta\sigma_D$ for the following loading:

- Most onerous fatigue train in one track plus FLM2 road load on one roadway girder.

Damage accumulation can be used for the following fatigue load combinations:

- Fatigue train on one track plus FLM3 road load on one roadway girder
- Fatigue trains in both tracks plus FLM3 road load on one roadway girder
- FLM3 road load on both roadway girders

Further details may be found in the Design Basis. The most onerous train is EC5. The fluctuating axial force range in the main cable due to the passage of an EC5 train is 10.6 MN. The main cable is anchored by 357 x 2 = 714 No M64 threaded anchor bars at the Calabria anchorage. Assuming a partial material factor for fatigue of 1.35 as specified by RFI Instruzione No 44F, the fluctuating stress in an anchor bar is therefore approximately:

$$\sigma_R = \frac{10.6 \times 1.35}{714 \times \frac{\pi}{4} \times 0.064^2} = 6.2\ MPa$$

In accordance with EN 1993-1-9 Table 8.1, an M64 threaded rod has a detail classification of 50 MPa, reduced by the size factor $k_5 = (30/64)^{0.25} = 0.827$. When further reduced in accordance with the S-N curves in RFI No 44F, it gives the following thresholds:

Reference stress	Symbol	Endurance (cycles)	Stress range (MPa)
Detail category (reference)	$\Delta\sigma_{C,r}$	2×10^6	41.4
Constant amplitude fatigue limit	$\Delta\sigma_{D,r}$	5×10^6	30.5
Cut-off limit	$\Delta\sigma_{L,r}$	1×10^8	16.7

Table 7-2 - Fatigue detail classification for anchor bars

By inspection the peak fluctuating stress range under a single EC5 train plus FLM2 road load will be considerably lower than the constant amplitude fatigue limit at 30.5 MPa. Therefore by inspection, the anchor rods have unlimited life.

The most onerous combination of fatigue loading is an EC5 train in both tracks plus an FLM3 road load vehicle on one roadway girder. This will result in a fluctuating stress range of approximately 13 MPa. This remains below the cut-off limit of 16.7 MPa. Therefore by inspection, no fatigue damage occurs under combined road/rail loading.

7.5 Anchorage crossheads

7.5.1 Introduction

The design requirement for the anchorage crossheads (crossheads) is to transfer the loads in the PPWS elements to the anchor block via the anchorage tendons, as stated above.

Initial sizing was through a simple hand calculation, looking at a simplified 2D problem. However, the eccentricity of the loading and supports is about equal to the thickness of the plate means ordinary beam theory is not appropriate and shear becomes the dominant stress transfer mechanism and a 3D volume analysis is necessary.

7.5.2 Modelling

7.5.2.1 Assumptions:

The following assumptions were made in modelling the crosshead:

- Reduce the full crosshead to a quarter, covering only a PPWS and tendon pair. This includes restraints along the lines of symmetry restricting displacement normal to the surface, and because the entire surface is restrained in one direction, rotation about the surface axis is also restrained (cf. a sliding cantilever support). This was confirmed during analysis which indicated that the stress distribution on the full model was limited to the volume between the prestressed tendon and the PPWS anchor rods - very little stress flow occurs between the areas attributed to each PPWS strand.
- No loss of contact between the crosshead and anchor wall. This is a reasonable simplification up to ULS since the PPWS strand load is lower than the applied prestress and loss of contact does not occur. This simplification enables a linear analysis to be carried out.
- Localised fillets are modelled to smooth out stress concentration around the perimeter of the bolt cut-outs as shown in Figure 7-2. This was found to be significant close to the anchor bars and has only been modelled locally where significant stress concentrations occur (see Figure 7-3).

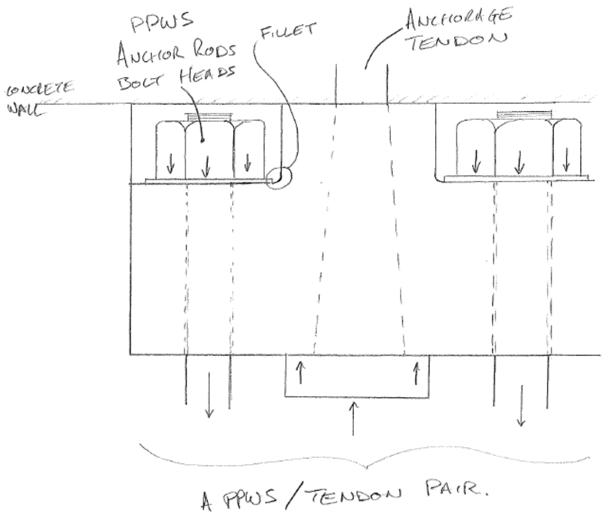

Figure 7-2: Section through a PPWS / Tendon pair of anchorage crosshead.

7.5.2.2 Mesh:

The crosshead plate is modelled as a linear volume, and not a surface, as explained above, due to the thickness of the crosshead compared to the plan dimensions (in the x-y plane shown in Figure 7-3). Due to the size of the model, the mesh has been concentrated where it is needed, around the anchor bolt heads, the fillets and the tendon bearing surface. The less refined (larger) mesh elements are 50mm per side tetrahedrons, and the more refined (smaller) mesh elements are 10mm per side tetrahedrons.

The loading is applied as surface loads where the PPWS anchor rod bolt washers sit, and where the anchorage tendon socket bears against the opposite surface. The restraint is in the form of a surface restraint, restricting the top surface (as viewed in Figure 7-3) of the crosshead from moving in the Z direction only.

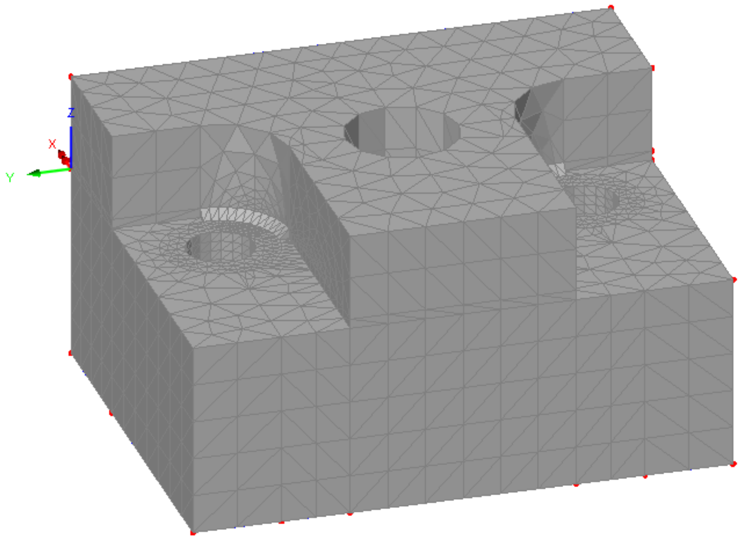

Figure 7-3: Anchorage Crosshead model, showing refinement of mesh around the anchor bars bolt heads.

7.5.2.3 Material:

The material used is a linear elastic steel. There is no yield stress; for all but hanger rupture, factored yield stress is the limiting von Mises equivalent stress, and so no section should go into yield. Young's Modulus of 209 GPa, and poisons ratio of 0.3.

7.5.3 Loading

There are 4 loading states to consider:

1. At SLS, the following limit states are defined:

a. Local stresses in the crosshead casting are required to remain everywhere below yield.

b. No loss of pre-compression occurs between the anchor wall and crosshead.

2. At ULS the following limit states are defined:

    a. Stresses in the crosshead casting may locally exceed yield provided the zone of plasticity is limited and does not affect structural performance or integrity. The attached PPWS strands are assumed to be at their ULS capacity.

    b. No loss of pre-compression occurs between the anchor wall and crosshead.

3. SILS: The limit states at SILS are the same as ULS, but with reduced partial material factors to reflect the increased allowable Damage level.

4. PPWS rupture: As an additional robustness check, the crosshead is designed to withstand the full breaking load of the PPWS strands. In this case, localised loss of precompression between the crosshead and the anchor wall is permitted. Localized plasticity is also permitted.

For each limit state, the applied PPWS load assumes 100% utilisation of the cable capacity, and thus uses the factored allowable stress in the main cable. I.e. at SLS, the fully utilised main cable load is at a factor of $\frac{1}{2.1}$ on the ultimate tensile load in the cable; therefore the applied load is $\frac{5.41}{2.1} = 2.58\ MN$ (see Table 7-3). In the case of SILS, the factored yield stress is 300MPa.

PPWS breaking load:

$$= \frac{\pi \phi^2}{4} \times N_{wires\ per\ strand} \times \sigma_{UTS}$$

$$= \frac{\pi \times 5.4^2}{4} \times 127 \times 1860$$

$$= 5410\ kN$$

$$= 5.41\ MN$$

The tendon is prestressed up to 0.75 x UTS (ultimate tensile strength):

$$Tendon\ Prestress = 0.75 \times A \times \sigma_{UTS}$$

$$= 0.75 \times 2850 \times 1.860$$

$$= 3980\ kN$$

$$= 3.98\ MN$$

Limit State	Reduction on PPWS breaking load	PPWS design force [MN]	Anchorage Tendon load [MN]
SLS	5.41 / **2.10**	2.58	3.98
ULS	5.41 / **1.67**	3.24	3.98
SILS	5.41 / **1.40**	3.86	3.98
ULS PPWS Rupture	5.41 / **1.00**	5.41	5.41

Table 7-3: Applied loads to Anchorage Crosshead.

7.5.4 Results

The utilisations ratio has been calculated as the resultant peak von Mises equivalent stress divided by:

$$\frac{\sigma_{yk}}{\gamma_m} = \frac{300}{1.05} = 286 MPa$$

Where σ_{yk} is the characteristic yield strength of the steel in the crosshead, and γ_m is the partial material factor.

Limit State	PPWS strand force applied [MN]	Peak von Mises Stress [MPa]	UR
SLS	2.58	213	0.74
ULS	3.24	213	0.74
SILS	3.86	251	0.84
PPWS Rupture	5.41	362	-

Table 7-4: Results of analysis of anchorage crosshead.

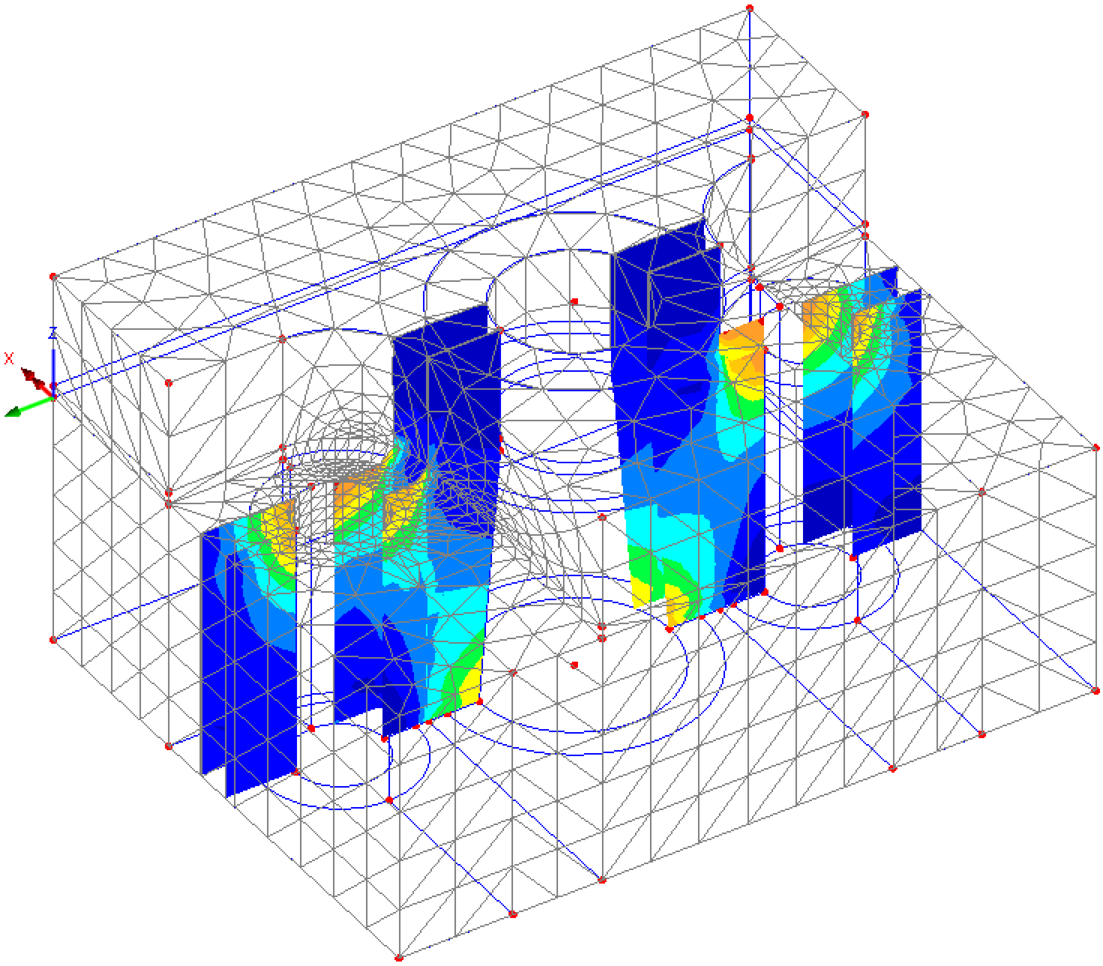

Figure 7-4: Von Mises equivalent stress contours, on slices through 3d showing location of Slice 1 and Slice 2 (Slice 1 is the one that runs through the center of the holes). The stress contours are for the Strand Rupture loadcase, with dark blue as zero stress, and dark red as 300MPa

7.5.4.1 SLS

There was shown to be no loss of precompression, and the peak stress in the crosshead was 213MPa (see Table 7-4), which is below the allowable stress. Figure 7-5 shows the von Mises stress distribution. Clearly the stress dissipates moving away from the loaded area.

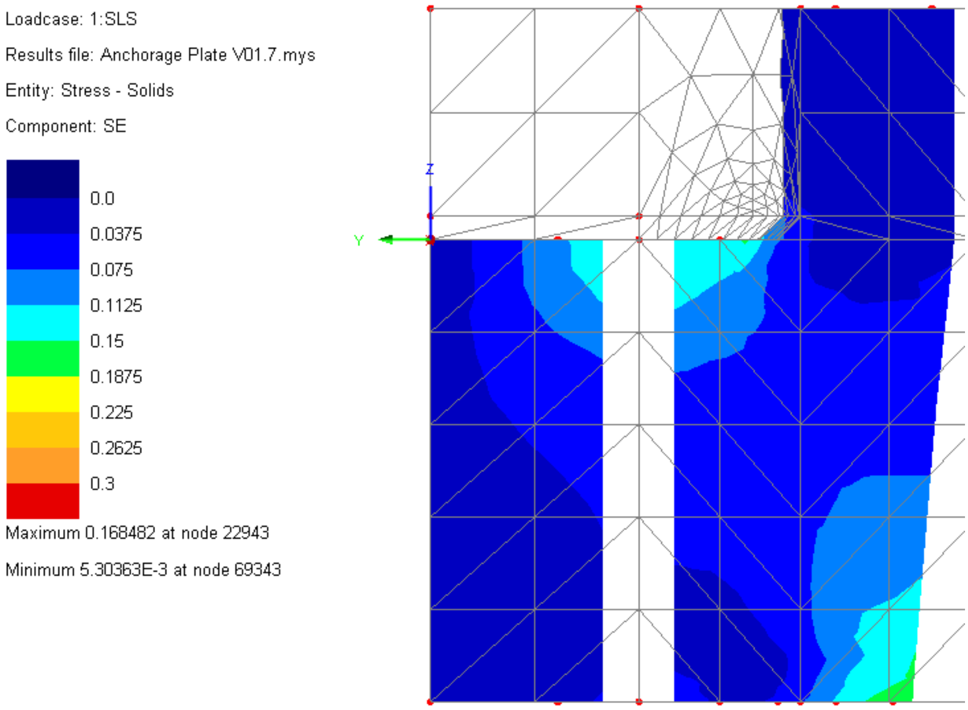

Figure 7-5: Von Mises equivalent stress contour plot for SLS loadcase. Slice 2 shown above, with the slice through the peak stress areas.

7.5.4.2 ULS

As with SLS there was no loss of precompression, and the peak stress in the crosshead was 213MPa (see Table 7-4), which is below the factored yield stress.

The reason the peak stress is the same in both SLS and ULS cases, is because the peak stress is caused by the prestress in the tendon, and not the load applied by the strands.

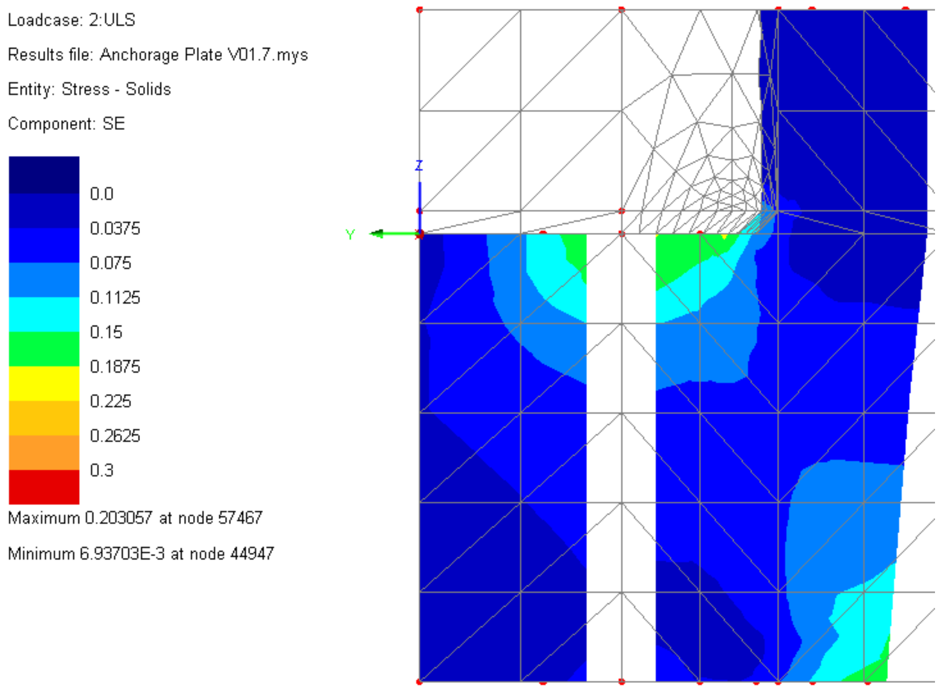

Figure 7-6: Von Mises equivalent stress contour plot for ULS loadcase. Slice 2 shown above, with the slice through the peak stress areas.

7.5.4.3 SILS

Under SILS there is no loss in precompression, and the peak stress in the crosshead, although caused by the PPWS load and not the tendon prestress, is below the capacity of 300MPa, resulting in a utilisation ratio of 0.84.

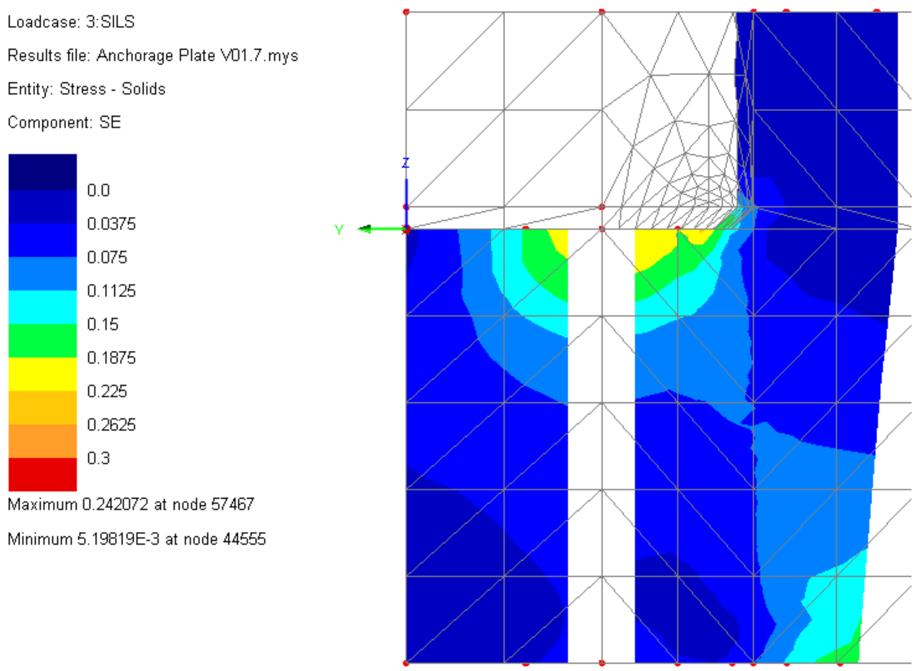

Figure 7-7: Von Mises equivalent stress contour plot for SILS loadcase. Slice 2 shown above, with the slice through the peak stress areas.

7.5.4.4 PPWS Rupture

PPWS rupture is the governing case for the capacity of the crosshead. As stated above, in this case localized plasticity is permitted, as long as it does not compromise the performance of the section. It is shown in the contour plots that whilst the peak stress of 362MPa will lead to plasticity, this is very localized, and the stress away from the concentration at the fillet is approximately 130MPa.

This is shown in Figure 7-4, Figure 7-10, and Figure 7-11, where the stress contours show the specific concentrations around the node through which the loads are applied. It is not so clear, but the area that go over yield are only shown in Figure 7-11, where there are 2 small black areas just under where the bolt washers would be seated. These small areas of stress in excess of yield, under this extreme load condition would redistribute so that the utilisation ratio is less than unity.

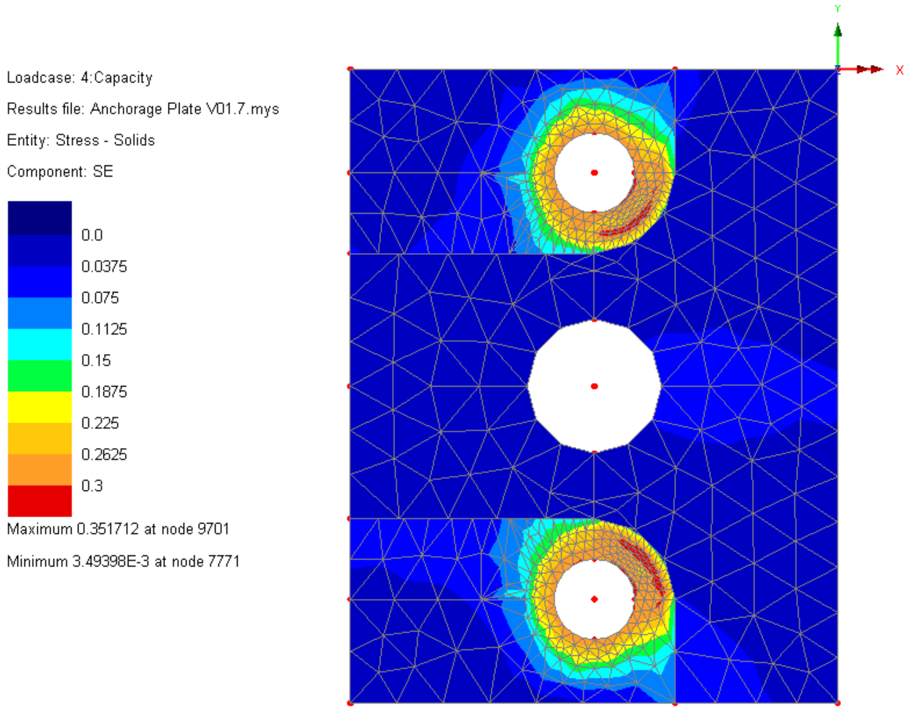

Figure 7-8: Stress plot from above showing stress concentrations around PPWS bolt washers at Strand rupture loadcase

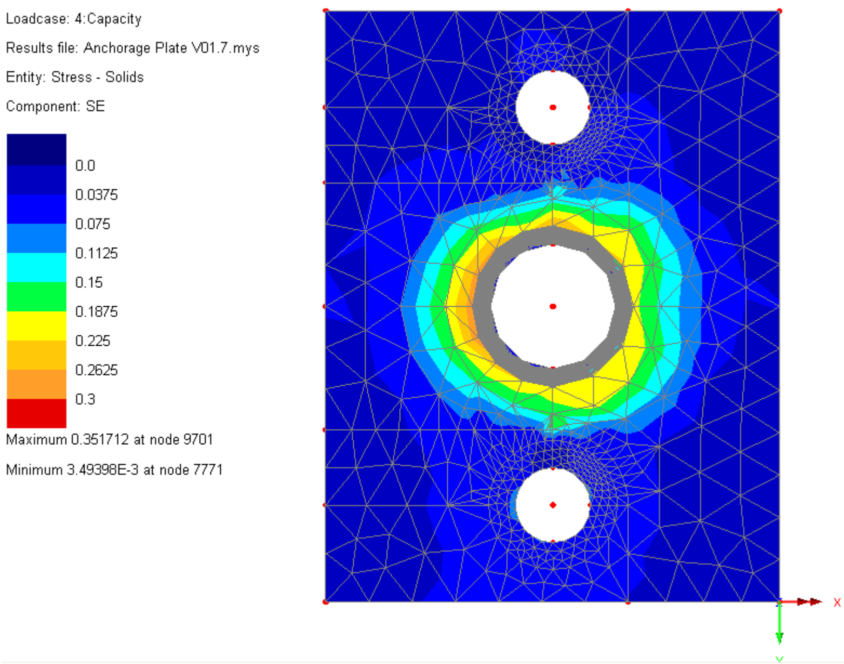

Figure 7-9: Stress plot from below (note axis), showing the stress distribution around the anchorage tendon bearing surface (an annulus around the hole in the centre). At Strand Rupture.

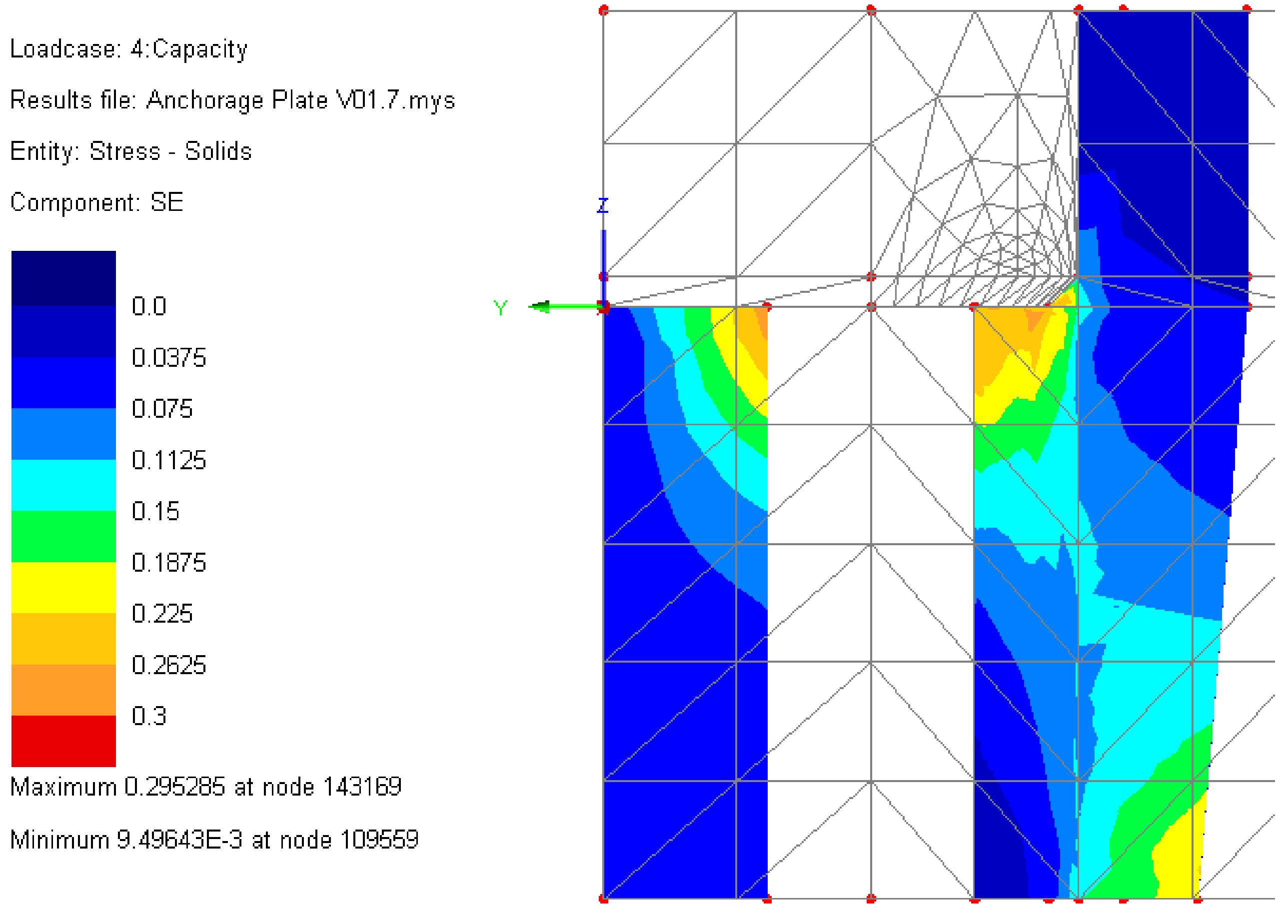

Figure 7-10: Von Mises equivalent stress contour plot for strand rupture loadcase. Slice 1 shown above, the slice through the centre of the holes. NB: yield is not reached on this slice.

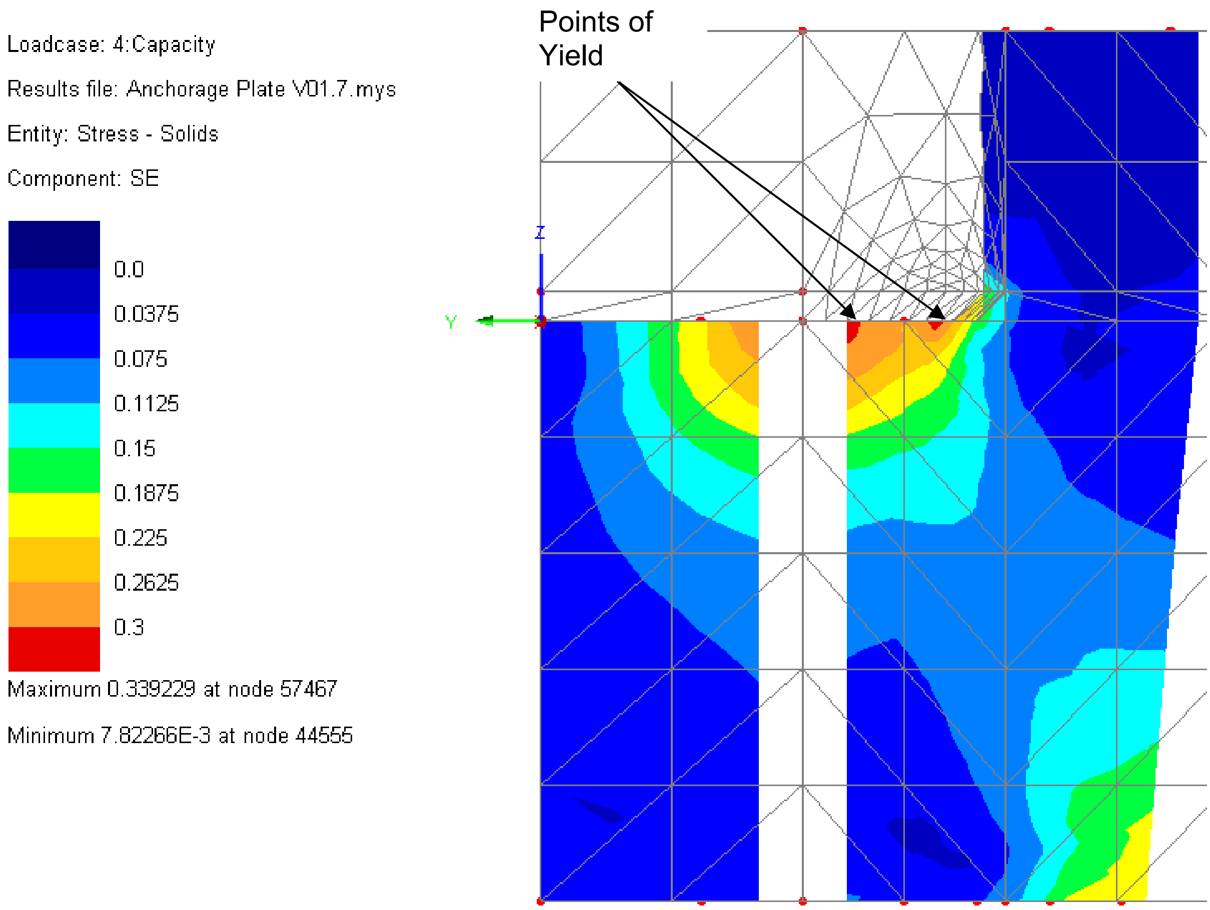

Figure 7-11: Von Mises equivalent stress contour plot for strand rupture loadcase. Slice 2 shown above, the slice through the peak stress areas. NB: fully black is above yield, there are 2 small places where this occurs - just to the left of the fillet, and at the corner of the bolt hole.

7.5.4.5 Conclusions:

At no stage does the von Mises equivalent stress go beyond the specified maximum for any limit state criteria. It has been shown that whilst there are small localized areas of plastically at Strand Rupture load, this is acceptable under the allowable Damage level and the performance of the crosshead is acceptable. This has been verified with a hand calculation showing that the shear area around a potential failure plug (plug shear) significantly exceeds the demand.

There is no lift off from contact with the concrete wall at SLS, ULS or SILS.