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Eurolink

PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

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INDICE

1 Introduction and main conclusion

The purpose of this report is to document the structural verification of the Calabrian anchor block. The verification shall prove the structure performs as defined for the Detailed Design.

The geotechnical verification of the Calabrian anchor block has been separately documented.

The structural verification of the Calabrian anchor block in the Definitivo Design phase is based on main cable forces from a global 3D FEM analysis model (IBDAS) and results from the soil-structure interaction obtained from a local 2D FEM ABAQUS analysis model.

The structural verification is carried out on a local 3D FEM IBDAS Model. The cable forces and the soil-structure interaction are applied as external loads to the local model. The loads of the local IBDAS Model are calibrated against the global IBDAS Model to ensure integrity between the different analysis models.

Supplementary calculations are performed by hand, e.g. for local stress zones and anchor chamber roof.

2 Project material

2.1 Codes

The analysis and verification of the Calabrian anchor block have been based on the listed standards and codes of practice:

- Design Basis referred to as [6]
- New Italian Code, chapter 4.1 Concrete Structures referred to as [4]
- Eurocode 2: Design of concrete structures referred to as [1] and [2]
- EN 10080: Steel for the reinforcement of concrete referred to as [5]
- VSL Post-Tensioning Systems referred to as [7]

2.2 Geometry

The geometry of the Calabrian anchor block is generally kept unchanged compared to Tender Design. Minor adjustments are performed in order to suit changes in the main cable geometry. The general shape of the Calabrian anchor block is shown in Figure 2-1.

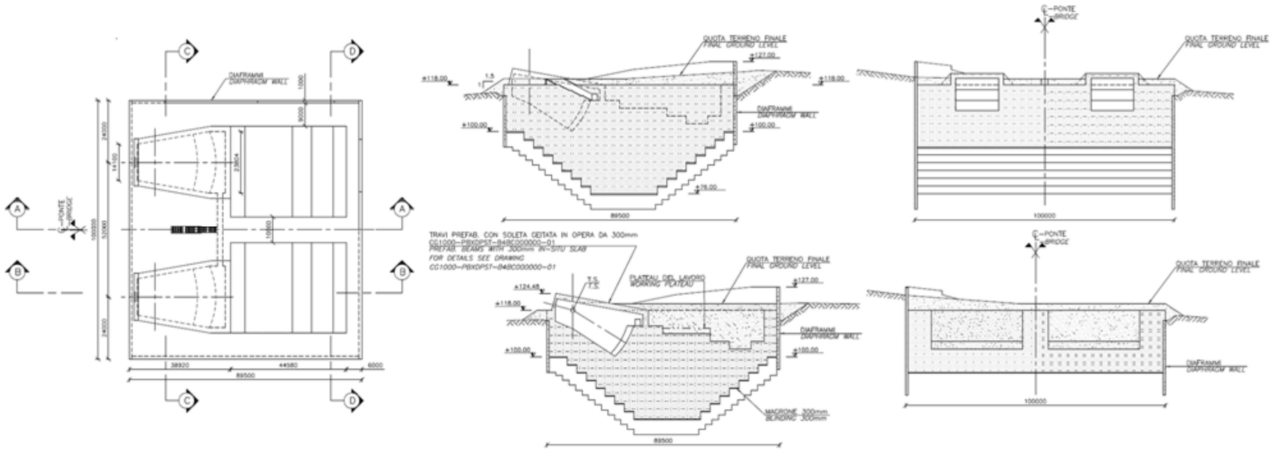

Figure 2-1 Plan and elevation view of the Calabrian anchor block

2.3 Materials

2.3.1 Concrete

Concrete C30/37 with the main properties as summarised in Table 2-1 is used in the calculations for the entire Calabrian anchor block.

Table 2-1 Concrete material properties according to [1].

Strength class	Characteristic compressive cylinder strength, f_{ck}	Design value of compressive strength, $f_{cd} = f_{ck}/ \gamma_C \cdot \alpha_{cc}$	Characteristic axial tensile strength, $f_{ctk,0.05}$	Mean tensile strength, f_{ctm}	Modulus of elasticity, E_{cm}	Maximal aggregate size, a_g
[-]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[N/mm^2]	[mm]
C30/37	30	17.0	2.0	2.9	33000	50
C35/45	35	19.6	2.2	3.2	34000	32
C45/55	45	25.5	2.7	3.8	36000	32

The partial safety factor for concrete is $\gamma_C = 1.5$; the coefficient taking account of long term effects on the compressive stress and of unfavourable effects resulting from the way the load is applied is $\alpha_{cc} = 0.85$ [4].

The concrete material behaviour is modelled by a parabola-rectangle relation with properties $\varepsilon_{c2} = 0.2\%$, $\varepsilon_{cu} = 0.35\%$, f_{cd} according to [1].

The tensile strength of concrete is neglected for structural verification.

Creep and shrinkage properties are considered in the local IBDAS model according to [1].

2.3.2 Reinforcement

Reinforcement B450 is used with the main properties as summarised in Table 2-2.

Table 2-2 Reinforcement material properties according to [1].

Strength class	Characteristic yield strength, f_{yk}	Characteristic strain at maximum force, ε_{uk}	Density, γ_c	Modulus of elasticity, E_s
[-]	[N/mm^2]	[%]	[T/m^3]	[N/mm^2]
B450	450	5.0	7.85	200000

The partial safety factor for reinforcing steel is $\gamma_C = 1.15$ for persistent and transient design situations [1].

The applied reinforcement material law is modelled by a linear-rectangular stress strain relation for compression and tension with the following properties: $\varepsilon_{yd} = f_{yd}/E_s$, $\varepsilon_{ud} = 3.5\%$, f_{yd}.

2.3.3 Prestressing

VSL Post-Tensioning tendons [7] with 19 strands are used for the anchorage of the main cables in the Calabrian anchor block with main properties as summarised in Table 2-3.

Table 2-3 Prestressing material properties according to [3].

Designation	Number of strands	Steel grade, $f_{p0,1k}$ /f_{pk}	Cross-section area, A_{pe}	Proof strength (0.1% proof stress), $F_{p0,1k}$	Characteristic braking load, F_{pek}	Modulus of elasticity, E_p
[-]	[-]	[N/mm^2]	[mm^2]	[kN]	[kN]	[N/mm^2]
EN10138-3 Y1860S7-15.7	12	1,670/1,860	1,800	3,006	3,348	205000
EN10138-3 Y1860S7-15.7	19	1,670/1,860	2,850	4,759	5,301	205000

The partial safety factor for prestressing steel is $\gamma_s = 1.15$ [1].

The relaxation loss is calculated according to [1], section 3.3.2. Ordinary relaxation properties (Class 1 relaxation) are assigned to the prestressing steel.

Relaxation of prestressing steel
according to EN 1992-2: 2005, 3.3.2

General Input

- Characteristic tensile strength of prestressing steel $f_{pk} := 1860 \frac{N}{mm^2}$

- Prestressing level $\mu := 0.75$ 2-1: 3.3.2 (7)

- Class of relaxation $class := 1$

class 1: wire or strand - ordinary relaxation
class 2: wire or strand - low relaxation
class 3: hot rolled and processed bars

- Time at which relaxation shall be calculated $t := 500000$ [hours] 2-1: 3.3.2 (8)

- Relaxation loss at 1000 days 2-1: 3.3.2 (7)

$$\rho_{1000} := \begin{array}{|l} 8 \ \text{ if } \ class = 1 \\ 1.25 \ \text{ if } \ class = 2 \\ 4 \ \text{ if } \ class = 3 \\ \text{"class?"} \ \ \text{otherwise} \end{array}$$

$\rho_{1000} = 8$ [%]

Relaxation loss

2-1: (3.28) - (3.30)

$$\rho_t := \begin{array}{|l} 5.39 \rho_{1000} \exp(6.7 \cdot \mu) \cdot \left(\frac{t}{1000}\right)^{0.75 \cdot (1-\mu)} \cdot 10^{-5} \ \text{ if } \ class = 1 \\ 0.66 \rho_{1000} \exp(9.1 \cdot \mu) \cdot \left(\frac{t}{1000}\right)^{0.75 \cdot (1-\mu)} \cdot 10^{-5} \ \text{ if } \ class = 2 \\ 1.98 \rho_{1000} \exp(8 \cdot \mu) \cdot \left(\frac{t}{1000}\right)^{0.75 \cdot (1-\mu)} \cdot 10^{-5} \ \text{ if } \ class = 3 \\ \text{"class?"} \ \ \text{otherwise} \end{array}$$

$\rho_t = 0.21$

$$\Delta\sigma_{pr} := \rho_t \cdot \mu \cdot f_{pk}$$

$\Delta\sigma_{pr} = 294 \frac{N}{mm^2}$

The relaxation loss of the prestress is $\Delta\sigma_{pr} = 294 N/mm^2$.

2.3.4 Ballast material

Sand with a density of γ_{Ba} = 17kN/m^3 is used as ballast material in the basins of the Calabrian anchor block.

2.4 Requirements and assumptions

2.4.1 Concrete exposure class

An exposure class XS1 (corrosion induced by chlorides as concrete surface is exposed to airborne chlorides) has been assumed for the entire Calabrian anchor block except the inside surfaces of the anchor chamber roof, see section 7.4.

2.4.2 Concrete cover

The concrete cover to reinforcement of the Calabrian anchor block is taken as 75mm to all surfaces in accordance with the concrete exposure class.

2.4.3 Basic hypotheses

The compatibility condition between reinforcement and concrete assumes perfect rigid bond ($\varepsilon_s = \varepsilon_c$). The hypothesis of Bernoulli is valid.

The concrete tensile strength is neglected in all limit states.

2.4.4 Structural damage levels

The following structural damage levels are assigned to the different limit states as summarised in Table 2-4.

The structural damage levels affect the utilisation of materials.

Table 2-4 Structural damage levels for anchor blocks [6].

Structural element	SLS	ULS	SILS
Anchor blocks	No damage 1) (ND)	Repairable damage 2) (RD)	Significant damage 3) (SD)
Anchor chamber roof	No damage 1) (ND)	Repairable damage 2) (RD)	-

1) All structural elements and restraint systems retain their nominal performance, remain in elastic state, and do not present any significant degradation due to fatigue.

2) Occurrence of localised inelastic behaviour which alters the overall performance capacities of the bridge.

3) Occurrence of inelastic behaviour which significantly alters the overall performance capacities of the bridge.

2.4.5 Prestressing loops

The verification of the anchor block assumes that all tendons in all prestressing loops are prestressed to 75% of the ultimate tensile strength, i.e. $0.75 \times 1860\text{N/mm}^2 = 1395\text{N/mm}^2$. This requires repeated prestressing of all tendons in all prestressing loops. The prestressing loops shall be stressed and grouted prior to commencement of the cable installation operations.

Prestressing loops are assumed to be tensioned simultaneously from both ends.

The short term tendon force is $P_{st} = 1395\text{N/mm}^2 \times 2850\text{mm}^2 = 3.975\text{MN}$.

The following assumptions have been made to estimate the long term losses:

- Relaxation: The long term loss due to relaxation has been estimated to $\Delta P_{pr} = 294\text{N/mm}^2 \times 2850\text{mm}^2 = 0.838\text{MN}$ according to [2], 3.3.2, see section 2.3.3.
- Creep: The concrete compression in the anchorage massive just behind the anchor plates after installation of the main cables and completion of the superstructure is very small. Therefore, the loss of prestressing force due to creep is evaluated to be negligible and has not been further investigated. $\Delta P_{cr} = 0$.
- Shrinkage: The long term shrinkage strain for large structure is a very uncertain value. However, an upper estimate of the strain can be taken as $\varepsilon_{cs\infty} = 0.028\%$. The long term loss due to shrinkage has been estimated to $\Delta P_{sh} = 0.028\% \times 205000\text{N/mm2} \times 2850\text{mm}^2 = 0.164\text{MN}$.

The resulting long term tendon force is $P_{lt} = P_{si} - \Delta P_{pr} - \Delta P_{cr} - \Delta P_{sh} = 3.975 - 0.838 - 0 - 0.164 = 2.973\text{MN}$.

2.4.6 Minimum reinforcement

The required minimum reinforcement has been calculated according to [2], section 7.3.2.

Required minimum reinforcement area
according to EN 1992-2: 2005: 7.3.2

Unless a more rigorous calculation shows lesser areas to be adequate, the required minimum areas of reinforcement may be calculated by 2-2:(7.1).

- Coefficients

$k_c := 0.4$ for rectangular sections and σ_c=0 (no axial load) 2-2: (7.2)

$k := 0.65$ for webs with h>=800mm or flanges with widths greater than 800mm 2-2: (7.1)

$f_{yk} := 450 \frac{N}{mm^2}$ $\sigma_s := f_{yk}$

Crack width neglected!
Maximum stress permitted in reinforcement immediately after formation of crack. This may be taken as the yield strength of the reinforcement, f_{yk}. A lower value, may, however, be needed to satisfy the crack width limits according to the maximum bar size or the maximum bar spacing. 2-2: (7.1)

- Concrete tensile strength (at the time of cracking)

$$f_{ctm} := \begin{pmatrix} 2.9 \\ 3.2 \\ 3.8 \end{pmatrix} \cdot \frac{N}{mm^2}$$

Concrete strength C30/C35/C45 EN 1992-1-1: Table 3.1

.

- Effective tensile strength of concrete

$h := 3\text{m}$ Height of a concrete block

Consider concrete age? Y/N $A := \text{"N"}$

$$t := \begin{cases} 28 & \text{if } A = \text{"N"} \\ 0.8\cdot h\cdot\frac{1}{\text{m}} + 1 & \text{otherwise} \end{cases}$$ Concrete age in days

$s := 0.2$ Coefficient for type of cement strength!
(here minimum value) EN1992-1-1: (3.2)

$$\beta_{cc\_t} := e^{s\cdot\left[1-\left(\frac{28}{t}\right)^{\frac{1}{2}}\right]} \qquad \beta_{cc\_t} = 1$$

$$\alpha := \begin{cases} 1 & \text{if } t < 28 \\ \frac{2}{3} & \end{cases} \qquad \alpha = 0.667$$

$$f_{ctm\_t_{i-1,0}} := \beta_{cc\_t}{}^{\alpha}\cdot f_{ctm_{i-1,0}} \qquad f_{ctm\_t} = \begin{pmatrix} 2.9 \\ 3.2 \\ 3.8 \end{pmatrix}\frac{\text{N}}{\text{mm}^2}$$

$$f_{ct\_eff_{i-1,0}} := \max\left(2.9\frac{\text{N}}{\text{mm}^2}, f_{ctm\_t_{i-1,0}}\right) \qquad f_{ct\_eff} = \begin{pmatrix} 2.9 \\ 3.2 \\ 3.8 \end{pmatrix}\frac{\text{N}}{\text{mm}^2}$$ 2-2: (7.1) & 7.3.2 (105)

- Required minimum reinforcement

$A_{ct} := 1$

$$A_{s\_min} := \frac{k_c\cdot k\cdot f_{ct\_eff}\cdot A_{ct}}{\sigma_s} \qquad A_{s\_min} = \begin{pmatrix} 1676 \\ 1849 \\ 2196 \end{pmatrix}\frac{1}{\text{m}^2}\text{mm}^2$$

A minimum reinforcement of $A_{s,min}$ = 1676mm^2/m^2 according to concrete C30/37 shall be applied for the Calabrian anchor block as general reinforcement.

The Calabrian anchor block is assumed to be cast in blocks of 3m × 11m × 26m with general reinforcement directed in all three dimensions, i.e. in longitudinal (s), transverse (y) and vertical (z) direction, respectively. General reinforcement is applied in the Calabrian anchor block resulting in reinforcement ratios ρ_{ss}, ρ_{sy} and ρ_{sz} as summarized in Table 2-5.

The proposed general reinforcement arrangement can be seen in Figure 2-2 and Figure 2-3. The general reinforcement is placed in groups of reinforcement 'walls' (planes assembled by vertical and longitudinal or transverse reinforcement bars). The distance within a group of reinforcement 'walls' in longitudinal (s) and transverse (y) direction of the bridge is 1.2m and 1.0m, respectively.

The distribution of reinforcement 'walls' in groups originates from the demand that no vertical reinforcement shall be placed in a distance of 1m from each block surface.

The vertical distance between horizontal bars is 200mm.

Table 2-5 Applied general reinforcement ratio for Calabrian anchor block.

Applied general reinforcement ratio	ρ_{ss}	ρ_{sy}	ρ_{sz}
	[mm^2/m^2]	[mm^2/m^2]	[mm^2/m^2]
Calabrian anchor block	2290	1874	2045

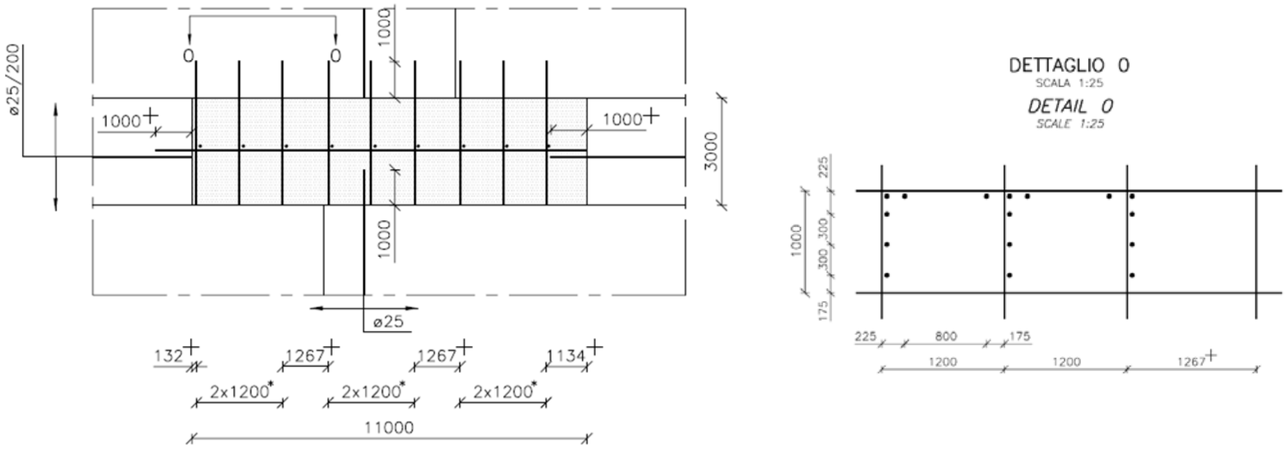

Figure 2-2 Arrangement of reinforcement 'wall' in global s-z-plane (longitudinal) for the Calabrian anchor block.

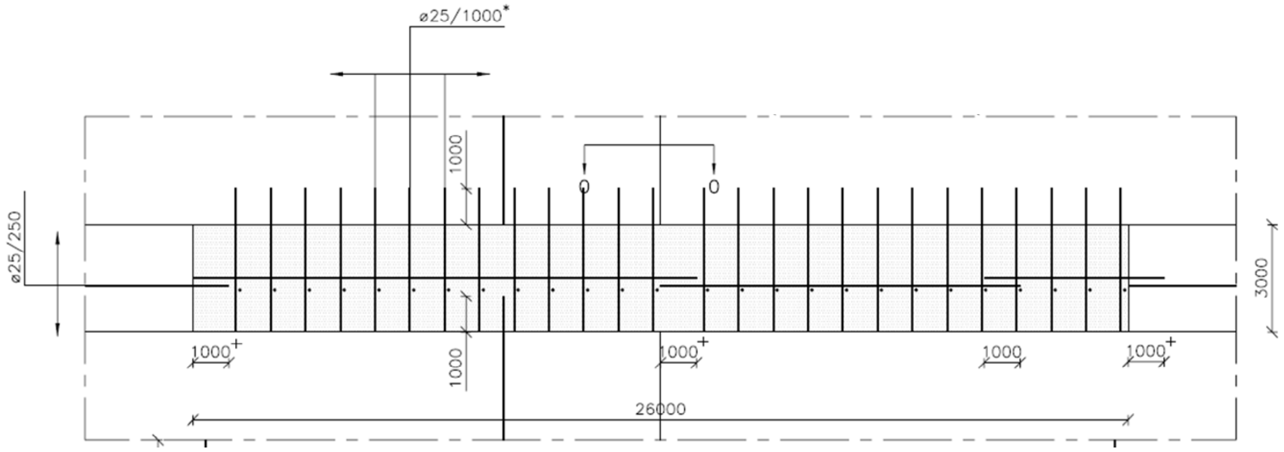

Figure 2-3 Arrangement of reinforcement 'wall' in global y-z-plane (transverse) for the Calabrian anchor block.

The dense minimum reinforcement based on a bar diameter of 25mm will intersect with the prestressing loops. It is assumed that in case a general or additional reinforcement bar intersects with a prestressing loop the reinforcement bar will be cut. The gap in the reinforcement bar will be closed by a new bar of sufficient length (including length of cut and lap length) placed in direct vicinity of the cut reinforcement bar.

2.4.7 Surface reinforcement

A general surface reinforcement of Ø25mm @ 250mm distance has been applied in both directions.

The surface reinforcement shall prevent extensive cracking of the concrete. This is assumed to be fulfilled, if the stresses in the surface reinforcement steel do not exceed 200N/mm^2 at the serviceability limit state.

2.4.8 Lap length

The lap length has been calculated for 25mm bars according to [1], section 8.7. The main calculation parameters and selected lap length for different reinforcement groups and different concrete strength are summarized in Table 2-6 to Table 2-9 for Ø25mm bars.

Table 2-6 Lap length for surface reinforcement.

Surface reinforcement				
Concrete strength	[-]	Concrete C30/37	Concrete C35/45	Concrete C45/55
Bond condition, η_1	[-]	0.7		
Ultimate bond stress, f_{bRd}	[N/mm^2]	2.1	2.31	2.84
Basic anchorage length, $l_{b,req}$	[mm]	1165	1059	863
Percentage of lapped bars relative to the total cross-section area, α_6	[-]	1.414 (50%)		
Lap length, l_0	[mm]	1200	1100	900

Table 2-7 Lap length for horizontal general reinforcement.

Horizontal general reinforcement				
Concrete strength	[-]	Concrete C30/37	Concrete C35/45	Concrete C45/55
Bond condition, η_1	[-]	0.7		
Ultimate bond stress, f_{bRd}	[N/mm^2]	2.1	2.31	2.84
Basic anchorage length, $l_{b,req}$	[mm]	1165	1059	863
Percentage of lapped bars relative to the total cross-section area, α_6	[-]	1.15 (33%)		
Lap length, l_0	[mm]	1000	900	750

Table 2-8 Lap length for vertical general reinforcement.

Vertical general reinforcement				
Concrete strength	[-]	Concrete C30/37	Concrete C35/45	Concrete C45/55
Bond condition, η_1	[-]	1.0		
Ultimate bond stress, f_{bRd}	[N/mm^2]	3.0	3.3	4.1
Basic anchorage length, $l_{b,req}$	[mm]	815	741	604
Percentage of lapped bars relative to the total cross-section area, α_6	[-]	1.5 (100%)		
Lap length, l_0	[mm]	900	750	700

Table 2-9 Lap length for additional structural reinforcement.

Additional structural reinforcement				
Maximum of horizontal and vertical general reinforcement.				
Concrete strength	[-]	Concrete C30/37	Concrete C35/45	Concrete C45/55
Lap length, l_0	[mm]	1000	900	750

3 Structural Analysis Models

3.1 General

The structural analysis comprises the following global and local models:

- Global IBDAS Model: 3D FE-Model of the entire bridge structure.
- Local ABAQUS Model: 2D FE-Model for determination of the soil-structure interaction.
- Local IBDAS Model: 3D FE-Model for structural verification.

A separate local analysis model exists for the Calabrian anchor block.

The structural verification is carried out on the Local IBDAS Model. The cable forces and the soil-structure reaction forces are applied as external loads to the Local IBDAS Model. The loads of the Local IBDAS Model are calibrated against the global IBDAS Model to ensure integrity between the different analysis models.

3.2 Global IBDAS Model

The global IBDAS model is described in detail in [8].

3.3 Local ABAQUS Model

The local ABAQUS model is described in detail in [9].

3.4 Local IBDAS Model

The local IBDAS model is described in detail in [10].

4 Loads and load combinations

4.1 Main cable forces (Global IBDAS Model)

The main cable consists of two cable planes (y-, y+). Each cable plane comprises two cables.

As can be seen from Figure 4-1, the main cable force P_C is deflected over the anchor saddle. The cable force P_A pulls at the anchorage wall and the anchor saddle is pressed on the anchor block with the anchor saddle force P_S. Both forces must be safely transferred into the anchor block.

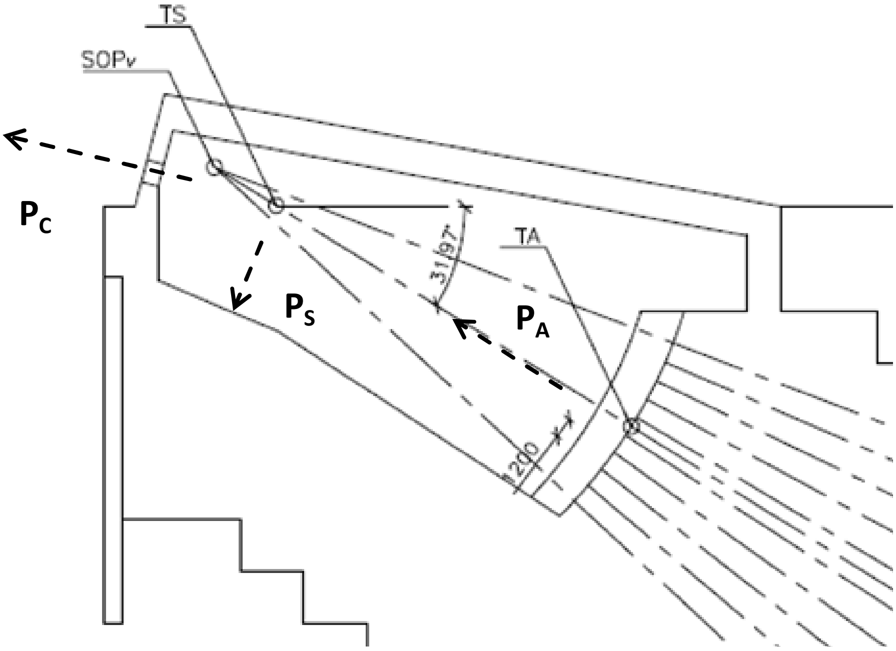

Figure 4-1 Resulting cable forces acting on Calabrian anchor block.

The design cable forces applied to the Local ABAQUS and Local IBDAS Models are summarized in Table 4-1. The design cable forces in Table 4-1 include forces from all four (two times two) cables.

The structural design verification has been carried out on the basis of the cable forces from the global IBDAS model version 3.3c. The results from the most recent global IBDAS model version

3.3f [8] are shown in parenthesis in Table 4-1 for comparison. The updated loads are approximately 2% less than the loads applied in the structural verification.

For each limit state and dynamic analysis type, the relevant IBDAS case (load combination) is given.

For comparison, the cable forces resulting from unfactored permanent loads (PP + PN) are as well given in Table 4-1.

Table 4-1 Design main cable forces for Calabrian anchor block dependent on the limit state and type of dynamic analysis.

Limit state	Type of dynamic analysis	IBDAS case (Global)	IBDAS criteria	Design cable force 1) 2), [MN]				
				Anchorage wall		Saddle		Total 3)
				ΣP_{As}	ΣP_{Az}	ΣP_{Ss}	ΣP_{Sz}	P_C
Permanent load 4)	static	1	all(fixed)	-2310 (-2273)	1437 (1415)	-310 (-306)	-698 (-690)	2722 (2679)
SLS 5)	static	6700	Min US	-2871 (-2834)	1787 (1764)	-371 (-367)	-841 (-832)	3377 (3334)
	seismic frequency spectrum analysis	6736	Min US	-3024 (-2959)	1881 (1842)	-391 (-384)	-888 (-871)	3557 (3481)
ULS (STR)	static	6500	Min US	-3524 (-3470)	2193 (2161)	-442 (-437)	-1008 (-996)	4138 (4077)
	seismic frequency spectrum analysis	6517	Min US	-3737 (-3625)	2327 (2258)	-474 (-460)	-1082 (-1051)	4391 (4260)
SILS	static	6800	Min US	-2894 (-2856)	1801 (1778)	-374 (-370)	-848 (-839)	3404 (3360)
	seismic frequency spectrum analysis	6812	Min US	-3295 (-3191)	2051 (1987)	-427 (-414)	-971 (-942)	3875 (3754)

1) Cable force positive (=tension) in positive global coordinate direction.

2) Values from global IBDAS model version 3.3c (values in parenthesis from global IBDAS model version 3.3f).

3) Total cable force is a tension force.

4) Permanent load for comparison; Permanent load = Total weight of structure for finished bridge (PP+PN).

5) SLS2 was found to be governing for the SLS limit state.

4.2 Soil-structure reactions (Local ABAQUS Model)

The soil-structure interaction has been calculated by the Local ABAQUS Model. As a result, the normal stress and shear stresses acting on the anchor block are obtained and are then applied to the anchor block in the Local IBDAS model.

An example for the normal and shear stresses acting on the anchor block at the ultimate limit state (IBDAS case 6500) are shown in Figure 4-2. The maximal normal and shear stress in Figure 4-2 are 0.846N/mm^2 and 0.382 N/mm^2, respectively.

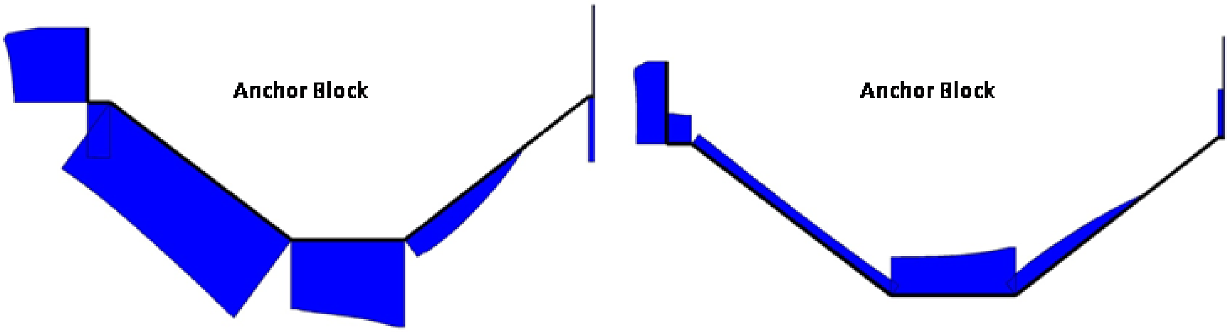

Figure 4-2 Normal stress (left) and shear stress (right) along the soil-structure interaction surface of Calabrian anchor block for ultimate limit state and IBDAS case 6500.

The complete soil-structure interaction calculation results can be found in [8] and [9].

5 Structural verification (Local IBDAS model)

5.1 General

The performance verification of the Calabrian anchor block has been carried out for the following cases:

- Construction phase 24 after construction of anchor block, installation of the prestressing cables and prestressing of all tendons to 75% of the characteristic tensile strength f_{pk}
- Serviceability limit state (SLS) of the final bridge
- Ultimate limit state (ULS) of the final bridge
- Structural integrity limit case (SILS) of the final bridge

For each case the following effects are evaluated (if relevant):

- Principal stresses for permanent loads (vector plots)
- Maximal/minimal principal stresses (contour plot)
- Utilisation ratio of concrete stresses
- Utilisation ratio of reinforcement stresses

The vector plots of the principle stresses indicate the main force flow in the massif anchor block and are important for the overall evaluation of the anchor block.

The contour plots of the maximal/minimal principal stresses highlight the overstressed areas in the anchor block.

The performance verification of the concrete and reinforcement has been carried out in two steps:

- In step 1, a general reinforcement as described in section 2.4.6 has been applied to the entire anchor block. The utilisation ratio of reinforcement (primarily) has been evaluated to localize areas with demand of additional reinforcement.

- In step 2, additional reinforcement has been applied in certain areas of the anchor block. The reinforcement area has been increased stepwise until the utilisation ratio of concrete and reinforcement fulfilled the design requirement.

The verification results are further discussed in the following sections.

5.2 Construction process

The vector plots of the principal stresses for the construction step under consideration shows together with the plots of maximal/minimal principal stresses that large minimum stress (compression) occur behind the upper anchorage. The minimum stress is found to be between -24.1 N/mm^2 and -27.2N/mm^2.

Compression stresses of some magnitude can also be found below the anchor saddle base plate. The local compressive stresses in the contact area with the anchor saddle base plate are further evaluated in section 6.2.

5.3 Serviceability limit state (SLS1 & SLS2)

The serviceability limit state 2 (SLS2) is governing over serviceability limit state 1 (SLS1). The cable forces are higher.

The local IBDAS model is not suitable to check the stresses in the surface reinforcement. However, based on the maximal principal concrete stresses (tension) which are in the magnitude of 2N/mm^2 at the surface and in most regions of the anchor block it can be concluded that the surface reinforcement in conjunction with the dense general reinforcement is suitable to limit the crack width..

The concrete tensile stress is in the magnitude of the mean tensile strength, i.e. 2.9N/mm^2, see Table 2-1. It can therefore be assumed that the crack width can be controlled to 0.2mm according to [4].

5.4 Ultimate limit state (ULS)

The plots of the maximal/minimal principle stress (contour plots) show that minimum stresses (compression) occur below the anchor saddle base plate. The minimum stress is found to be approximately between -15.2N/mm^2 and -17.3N/mm^2. The minimum stress behind the upper anchorage zone is between -12.1N/mm^2 and -14.0N/mm^2.

The utilisation ratio of the concrete is defined as the stress in the concrete divided by the design value of concrete compressive strength.

The utilisation ratio of the concrete stresses at the ultimate limit state is well below 1.0 in almost all zones of the anchor block except in local areas below the anchor saddle base plate and the upper anchorage zone. In these locals zones utilisation ratios above 1.0 were found. Following, the concrete strength should be locally increased, see section 5.6.

The utilisation ratio of the reinforcement is defined as the stress in the reinforcement divided by the design yield strength of the reinforcement.

The utilisation ratio of the reinforcement is first found for intensity equal to general reinforcement (step 1). Additional reinforcement has been applied in the over utilised zones of the anchor block and the updated utilisation ratio has been found. Only minor local areas remain over utilised which is found to be acceptable as not all calculation reserves have been utilised, e.g. the redistribution of stresses or the increased concrete strength in some zones of the anchor block. The main zones where additional reinforcement is required are identified and sufficiently covered by additional reinforcement. These zones are in agreement with force flow obtained from the principle stresses (vector plot). The structural analysis model is capable to capture the global force flow. Local zones have to be designed by hand and engineering judgement.

The final reinforcement intensities in the anchor block are included in Appendix E.

5.5 Structural integrity limit state (SILS)

The structural integrity limit state is not governing over the ultimate limit state. The cable forces are lower, see Table 4-1.

5.6 Design conclusions

The compressive concrete stresses are highly utilised below the anchor saddle base plate and in the anchorage zone. These areas are also highly reinforced with a dense reinforcement. Therefore, an increased concrete strength of C35/37 is chosen in the front part of the anchor block, see 2.3.1.

It was further decided to increase the concrete strength locally in the anchorage zone and below the anchor saddle to C45/55, because of overutilization of the concrete compression at the ultimate limit state.

Thereby, the casting and compacting of concrete will as well be improved by a smaller maximal aggregate size, i.e. 32mm instead of 50mm.

6 Supplementary verification

6.1 Main cable anchorage

6.1.1 Design forces from main cable

The design cable forces per cable plane (y-, y+) in the saddle P_S and in the anchorage wall P_A resulting from Table 4-1 are summarized in Table 6-1 for the Calabrian anchor block.

Table 6-1 Design forces P_A and P_S from main cable P_C for Calabrian anchor block dependent on the limit state.

Limit state [1]	Maximal anchorage wall force/per cable plane, P_A [MN/cable plane]	Maximal anchor saddle force/per cable plane, P_S [MN/Saddle]
Permanent load	1360 (1339)	382 (377)
SLS	1780 (1743)	485 (476)
ULS (STR)	2199 (2135)	590 (574)
SILS	1939 (1880)	530 (514)

[1] Values from global IBDAS model version 3.3c (values in parenthesis from global IBDAS model version 3.3f).

6.1.2 Prestressing loops

The cable force in the anchorage wall per cable plane P_A transfers large tension forces into the anchor block. Therefore, the anchor block is widely prestressed by a large number of prestressing loops. The distribution of the prestressing loops per cable plane follows from Figure 6-1.

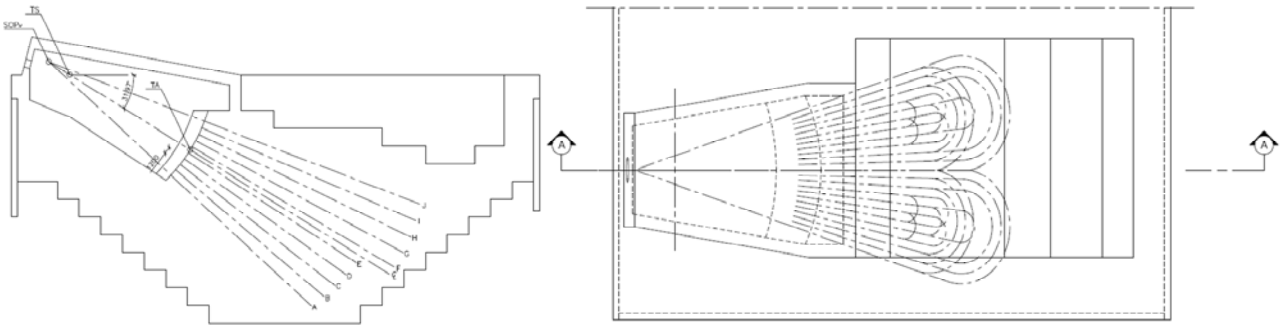

Figure 6-1 Arrangement of prestressing loops in the Calabrian anchor block.

It is important that the anchor plates will not be lifted from the concrete surface (split). The prestressing force shall therefore also be checked after long term losses. The lifting condition is considered to be fulfilled, if the average cable force max P_A divided by the number of prestressing tendons is smaller than the prestressing force after long term losses P_{lt}, i.e. 2.973MN, see section 2.4.5.

The maximal cable force in the anchorage wall per cable plane results from the ultimate limit state, i.e. max P_A = 2199MN, see Table 6-1. The cable force P_A (per cable plane) is transferred into the anchor block by 756 tendons. The resulting force per prestressing tendon is 2199MN/756 = 2.909MN, i.e. smaller than P_{lt} = 2.973MN.

This shows that lifting of the anchor plates from the concrete is prevented.

6.1.3 Anchor zones

In the anchorage zone the prestressing tendons are anchored in crossheads, see Figure 6-2. Different crossheads exist for different numbers of tendons connected to one crosshead.

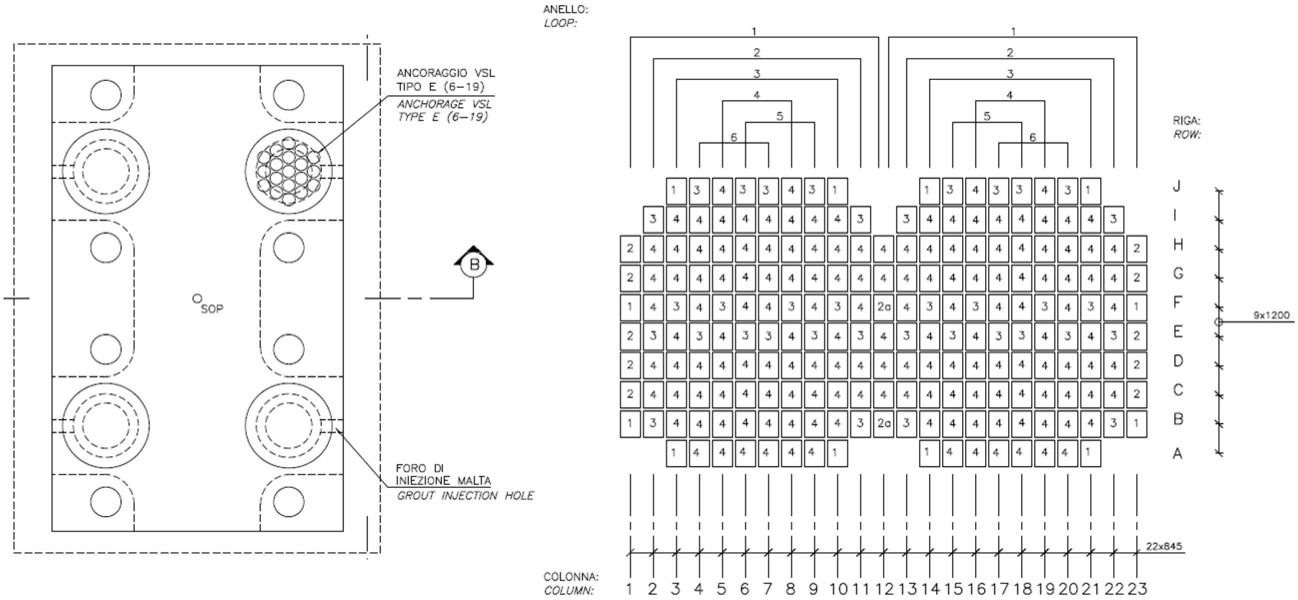

Figure 6-2 Crossheads for four tendons and overview of general tendon and loop distribution

Spiral reinforcement Ø16mm with 7 pitches, a pitch height of 60mm and a diameter of 400mm is provided in the anchorage zone as local zone reinforcement.

In general, one prestressing tendon is used to anchor each PPWS cable in the anchor block. Thus, the breaking load of the prestressing tendon, see Table 6-2.

Table 6-2 Design check of anchorage zone.

	Prestressing tendon, [MN]	PPWS cable, [MN]
Breaking load	5.301	5.410

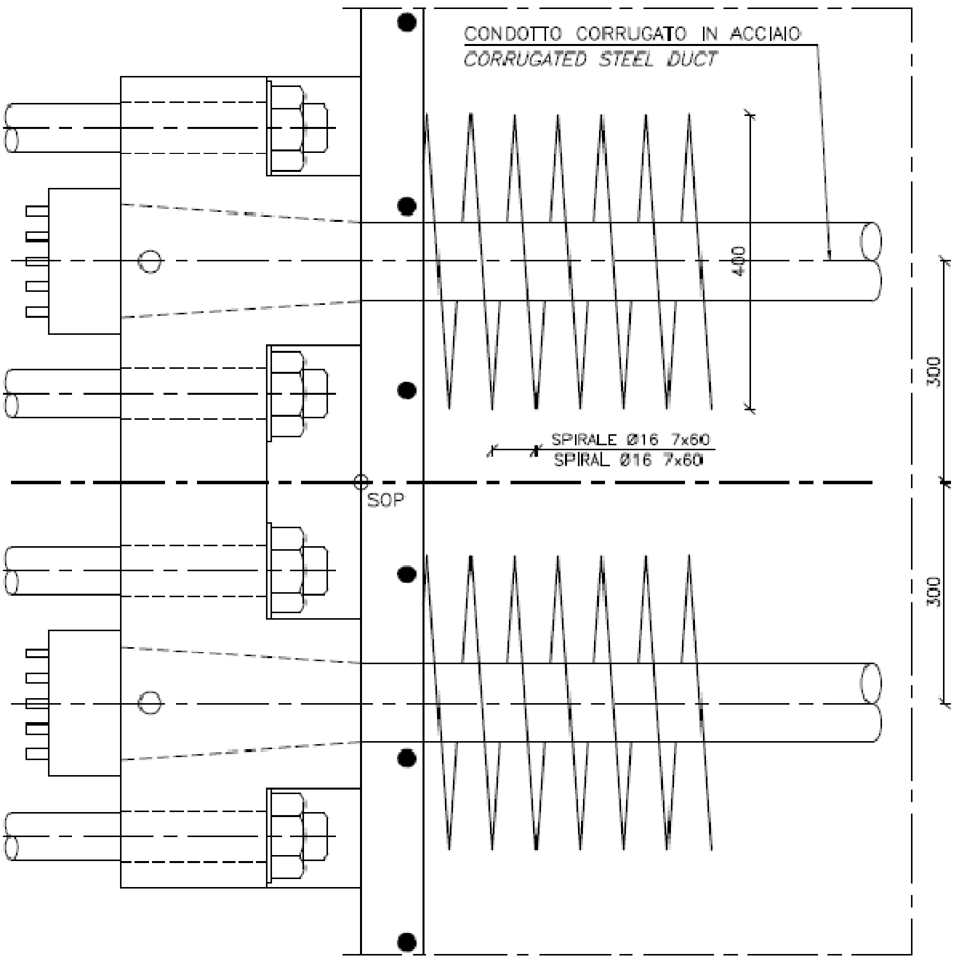

Figure 6-3 Connection of PPWS cables on anchor block.

6.2 Anchor saddle support

The anchor saddle is supported by the anchor block in the anchor chamber. The saddle force is transferred to the concrete through the anchor saddle base plate, i.e. 2.75m × 8.5m. The maximal force in the anchor saddle results from the ultimate limit state, i.e. max P_S = 590MN, see Table 6-1.

The concentrated resistance force F_{Rdu} for a uniform distribution of load on the anchor saddle base plate is F_{Rdu} = 765MN according to [1], 6.7. The concrete strength below the anchor saddle base plate used in the calculation is C30/37.

The concentrated resistance force F_{Rdu} is larger than the maximal anchor saddle force resulting from the ultimate limit state, i.e. $F_{Rdu} = 765MN > 590MN = P_{S,max}$, see Table 6-1. Thus, local crushing of concrete is prevented.

Tension reinforcement is required in all three global directions, because of the inclination of the anchor saddle force in the global s-z-plane. Transverse tension forces are estimated to 11.8MN/m and 7.6MN/m acting parallel to the anchor saddle base plate edge of 2.5m and 8.5 lengths, respectively. The tension reinforcement needs to be distributed over a height of 7m.

The transverse tension reinforcement is placed perpendicular to the anchor saddle force over a depth of 1.0m. Below 1.0m, the dense additional and general reinforcement below the anchor saddle base plate is utilised to further distribute the anchor saddle force in the anchor block.

6.3 Surface reinforcement

The surface reinforcement shall prevent extensive cracking of the concrete. This is assumed to be fulfilled, if the stresses in the surface reinforcement do not exceed 200N/mm^2 at the serviceability limit state, see section 5.3.

7 Anchor Chamber Roof

7.1 General

The design of the anchor chamber roof has been developed as precast concrete elements forming a formwork for a 0.3m thick concrete slab on top. The total height of the anchor chamber roof structure is limited to 2m. Hence, the precast elements are chosen to be 1.7m height.

The precast elements span over the short length of the anchor chamber in transverse direction of the bridge axis. The length of the precast elements is considered to be 600mm longer than the span length of the precast elements as summarized in Table 7-1.

The precast elements will be supported by either the massive anchor block structure or 2m thick walls. In order to do so, a recess of 2.1m height and 2m depth has to be provided.

The anchor chamber roof of the Calabrian anchor block is inclined by 11°. It is assumed that the precast elements are placed besides each other with the same inclination. Therefore, the loads create a biaxial loading state.

Precast elements and slab form a composite section meaning that they work as one cross-section after hardening of the concrete slab.

The slab is also structurally connected with the adjoining vertical walls or the massive block structure at the supports, respectively. Hence, a fixed support condition can be assumed in this state. Thereby, the air tightness of the anchor chamber is secured (dehumidification system) and the structural effectiveness is enhanced.

The objective of the design is to define a cross-section that is suitable for the entire anchor block roof. Precast element 18 (or 18-15) is governing for the geometry of the precast elements. Precast element 18 is the longest element with the highest loads applied to.

The precast elements are numbered from #1-18; precast element #1 being the shortest and element #18 being the longest.

Table 7-1 Precast elements span length of Calabrian anchor block.

Precast element #	Span length	Total length	Precast element #	Span length	Total length
[-]	[m]	[m]	[-]	[m]	[m]
1	15.4	Span length + 0.6	10	21.2	Span length + 0.6
2	16.1		11	21.8	
3	16.7		12	22.5	
4	17.3		13	23.1	
5	18.0		14	23.7	
6	18.6		15	24.4	
7	19.3		16	24.4	
8	19.9		17	24.4	
9	20.5		18	24.4	

7.2 Geometry

The selected cross-section and dimensions follow from Figure 7-1.

The precast elements are prestressed with a parabolic shaped tendon. The tendon is positioned 1.2m and 0.2m from the bottom of the precast element at the support and at midspan, respectively.

The local coordinate system s'/z' is defined as shown in Figure 7-2.

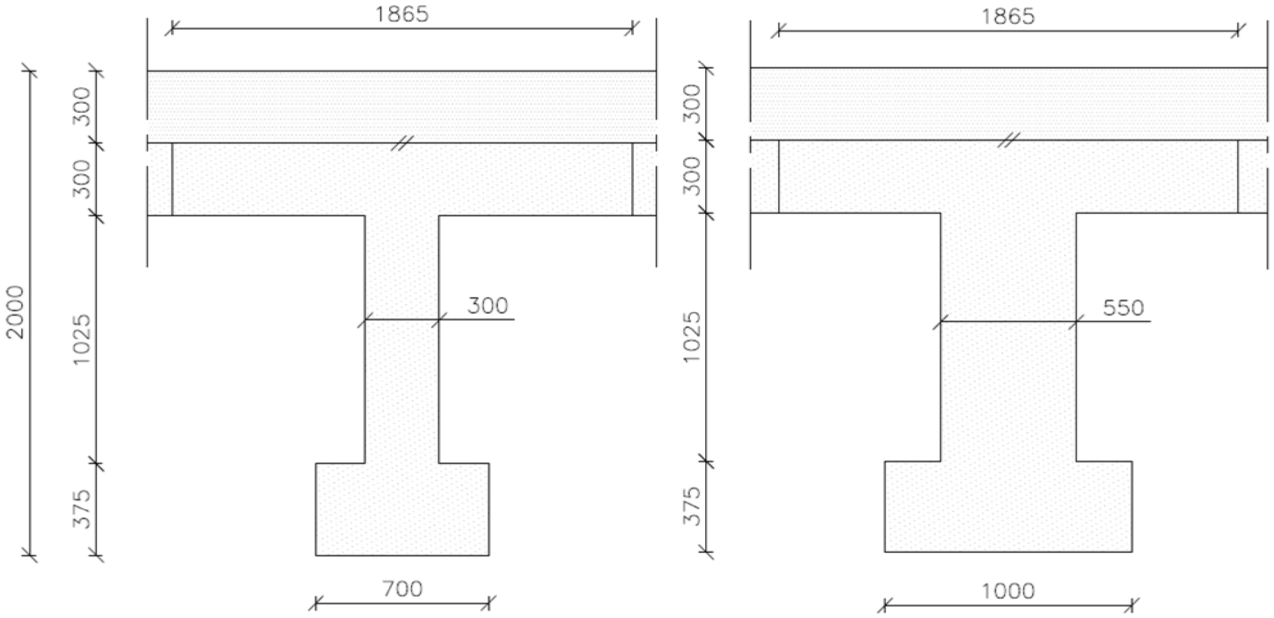

Figure 7-1 Cross-section of precast elements at midspan (left) and at support (right).

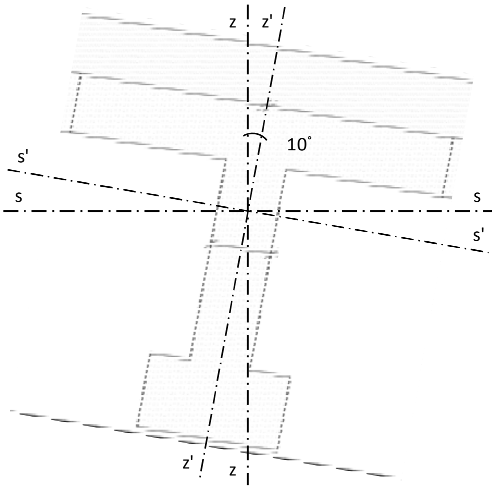

Figure 7-2 Local coordinate system.

7.3 Materials

Materials and partial safety factors have been applied for the anchor chamber roof as specified in section 2.3.

Concrete of class C45/55 has been used for the precast elements and the slab, see Table 2-1.

Prestressing tendons according to Table 2-3 have been applied.

7.4 Requirements and assumptions

Exposure class XC1 (low air humidity in the anchor chamber) has been assumed for the precast elements of the anchor chamber roof.

The concrete cover to reinforcement of the precast elements is taken as 45mm to all surfaces in accordance with the concrete exposure class.

7.5 Loads and load combinations

7.5.1 Permanent actions (PN and PP)

The average characteristic self weight of the precast elements (PN) is $g_{1,k}$ = 29kN/m.

The characteristic self weight of the 300mm thick slab (PN) is $g_{2,k}$ = 14kN/m.

The anchor chamber roof has to be designed to carry a 3m thick sand ballast layer (PP). The considered characteristic sand ballast load is $q_{B,k}$ = 112kN/m in wet condition with a unit weight of 20kN/m^3.

7.5.2 Seismic action (VS)

The anchor chamber roof and its parts will be excited by the anchor block in case of an earthquake. The design seismic motion is defined through the response spectra of the horizontal and vertical components [6].

For the design of the anchor chamber roof, only the vertical component of the spectra has been considered. Furthermore, seismic action has only been considered for the final state, see Table 7-4. In the final state, all permanent loads will be excited.

Peak ground acceleration levels of 2.6m/sec^2 and 5.7m/sec^2 at the serviceability and the ultimate limit state are assumed according to [6].

7.5.3 Miscellaneous

An additional characteristic load for personal hand tools (CL) has been considered, i.e. $q_{CA,k}$ = 1.9kN/m.

Precast element 18 is post-tensioned by a tendon with 19 strands. The tendons are prestressed to 75% of their characteristic tensile strength f_{pk}. The overall time dependent losses are estimated to 15%. The resulting prestressing force assumed in the calculation 2.939MN.

7.5.4 Load combinations

Load combination 1 and 3 were considered for the design of the anchor chamber roof at the serviceability and the ultimate limit state according to [6]. Load combinations and according load safety factors are summarized in Table 7-2 and Table 7-3.

Combination 3 includes the seismic action which can be directed downwards or upwards. In the first case the permanent loads act adverse and in the second case the permanent loads act beneficial).

Table 7-2 Load combinations and according load safety factors considered at the ultimate limit state.

Combination	PP	PN	VS	CL	Prestressing
1	1.25	1.5	-	1.0	1.0
3 (adverse)	1.25	1.5	1.0	-	1.0
3 (beneficial)	0.95	0	1.0	-	1.0

Table 7-3 Load combinations and according load safety factors considered at the serviceability limit state.

Combination	PN	PP	VS	CL	Prestressing
1	1.0	1.0	-	1.0	1.0
3 (adverse)	0	1.0	1.0	-	1.0
3 (beneficial)	1.0	1.0	1.0	-	1.0

7.6 Structural analysis

The structural analysis assumes that the loads are transferred in direction of the precast elements.

The structural analysis has to consider different structural systems at different stages. The design of the anchor chamber roof considers the final state of the precast elements (Final Beam) and the final state of the anchor chamber roof (Final). Table 7-4 summarises the static system and the considered loads for each state.

Table 7-4 Static system

State	Static system	Load
Final Beam	Simply supported beam	PN, PP, Prestressing, CL
Final	Beam fixed at supports	PP (sand ballast), CL, VS

The inclination of the anchor chamber roof towards the main pylons results in a biaxial loading of the static system.

The resulting bending moment distribution at the serviceability and the ultimate limit state can be found in Appendix F. The prestressing action is taken into account separately.

The seismic action in Combination 3 (beneficial) results in a positive bending moment at the support, see Appendix F.

The governing design shear force including prestressing results from load combination 3 (adverse) and the final state. The design shear force is $V_{z',d}$ = 3.153MN at position x=1.0m.

7.7 Structural verification

7.7.1 Design criteria

The design of the anchor chamber roof focuses on check of the bending moment and shear capacity (only in local vertical direction) at the ultimate limit state and the crack width limitation at the serviceability limit state. The maximal crack width for cracks is 0.2mm according to [4].

Besides, reinforcement has been designed according to [1] to secure the composite action between the precast element and the slab on top and the fixation of the slab in the massive anchor block structure at the supports as considered in the structural analysis.

7.7.2 Design check results

The design solution fulfils all the above mentioned design criteria. The result of the design provides a cross-section that can be used for all precast elements and for both anchor blocks.

The design of the precast elements reveals that prestressing is required for an efficient design solution. Considering shorter precast elements, the number of strands in the prestressing tendons can be reduced.

Therefore, precast elements #10-18 are prestressed with one tendon of 19 strands, #3-9 are prestressed with one tendon of 12 strands and #1-2 do not require prestressing.

The seismic action and the high sand ballast load determine the design of the precast elements. The web and the bottom flange of the precast element have to be widened at the support over a length of 1.2m from the end.

8 Reference

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Appendix A: Slice numbering

Appendix B: Construction phase 24

Appendix C: SLS

Appendix D: ULS

Appendix E: Reinforcement intensities

Appendix F: Anchor chamber roof