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)
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INDEX

1 Executive Summary

1.1 Introduction

This report describes the design of the following structural elements of the terminal structures:

- Piers
- Foundations
- Cross beam
- Tie-down diaphragm

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 transverse diaphragm walls underneath the foundation have been removed.
- The foundation slab dimensions have changed compared to the tender design (26 m x 70 m maximum for Sicilia side and 21 m x 70 m for Calabria side, excluding the diaphragm walls around the excavated area).
- The pier elevations have been modified accordingly the changing made in the master profiles and in the slopes of the suspended bridge.
- The pier section has been subdivided into two different sections (one with thickness of 0.80 m and the other with of 0.50 m).
- An additional diaphragm adjacent to the tie-down diaphragm has been added, together with a diaphragm in correspondence of the bottom of the slab of the beam connecting the piers.
- The cross beam dimensions and the pier dimension above the diaphragm have been reduced.

The layout of the Terminal Structure is shown in Figure 1-1.

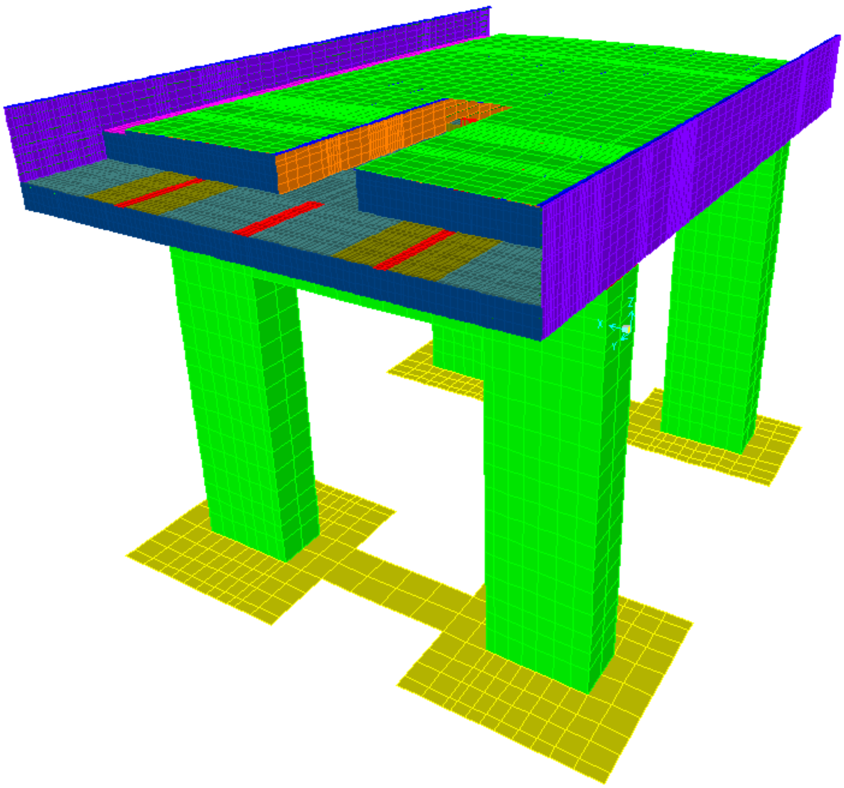


Figure 1-1: General layout

The substructures consist of a two piers and a cross beam which are founded on connected concrete slabs. Each pier is 2-cell structure and the cross-beam a single box structure.

A view of the substructure is shown in Figure 1-2.

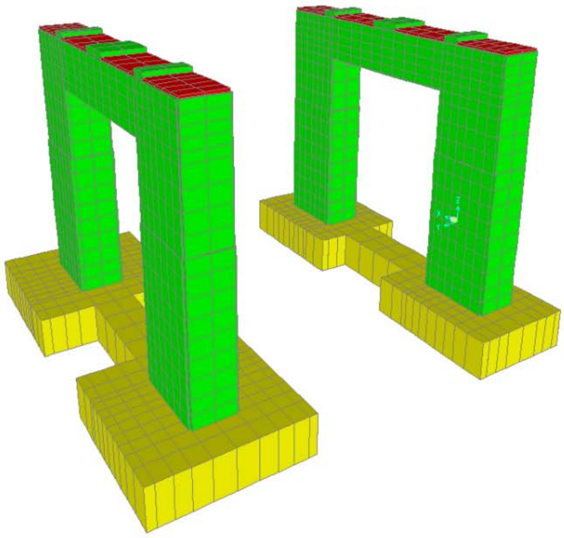


Figure 1-2: Side view of substructures

1.2 Scope

This report describes the design of the structural elements of the Terminal Substructures, both Sicilia and Calabria side.

The Specialist Technical Design Report is summarising the design which is verified in detail in the Design Report (Report CG1002-P-CL-D-P-SV-S8-00000000-01).

1.3 Materials

Structural concrete is of Grade C32/40 and according to EN 206-1, for the foundations and Grade C40/50 and according to EN 206-1 for the piers. Maximum aggregate dimension is 25 mm.

Reinforcement are of carbon steel Grade B450C and according to EN 10080.

1.4 Structural Analysis

The Terminal Structures are modelled and analysed by use of a SAP2000 model whilst the global Messina Strait Bridge was analysed in the COWI proprietary analysis program IBDAS (Integrated Bridge Design and Analysis System).

Bearing reactions for selected fixed loads from the IBDAS model have been applied to the SAP 2000 model together with bearing reactions from the adjacent viaducts, to reflect the interaction between the Terminal Structure and the Suspended Deck.

The structural analysis takes into account the construction sequence of the Terminal Structure.

1.5 General Description

The Terminal Structures are the approach infrastructures connecting the suspension bridge with the existing road and railway network on the two sides.

On the Sicilia side the Terminal Structure is linked to the suspension bridge and to the Pantano Viaduct; on the Calabria side it is linked to the suspension bridge and to the access viaduct.

The suspension bridge lands on a number of bearings on the Terminal Structure where a trench is provided for the rail girder which extends longer into the Terminal Structure than the roadway girder.

The Pantano viaduct also lands on bearings provided for its geometry and for both the viaduct and the suspension bridge the Terminal Structure is prepared for installation of large expansion joints which at the suspended deck interface accommodate approximately ± 2000 mm.

The Terminal Structure is a composite structure consisting of a concrete deck on a steel box structure. In plan the overall dimensions of the Terminal Structure are 60.870 m and 94.200 m for the width and length respectively, Sicilia side and 60.870 m and 94.200 m for the width and length respectively, Calabria side; whilst the height varies between 2.810 m at the Suspended Deck support and 10.800 m at the facade. The concrete deck slab has a thickness varying between 300 mm and 400 mm including 50 mm of pre-cast concrete elements (also referred to as predalles).

The deck supports 2 railway tracks, 3 roadways (of which one is an emergency lane) and 2 service lanes.

The service lane is located on the outside of the roadway over the entire length of the Suspended Deck and the Terminal Structure. It is the primary access route for inspection and maintenance.

Along the railway is a platform located on either side of the railway girder for evacuation if necessary. The platform is continuous over the entire Terminal Structure length.

Both terminal structures have just one span with a cantilevered overhang of 19.600 m at each end. They are connected to the suspended cable system by hanger cables, which - via an opening through the superstructure of the Terminal Structure - are tied-down into the substructure.

Bearings on each substructure support the superstructure vertically and transversely while a number of bearings support the Suspended Deck and the adjacent Pantano Viaduct.

Piers are made of reinforced concrete with a 2-cell box section with a decreasing thickness above the diaphragm. At the top, they are connected with a reinforced concrete beam section (28 m x 7 m), with a section conformed to allow jack housing.

Foundations are direct on concrete slabs. They comprise an excavation area of 73 m x 76.5 m for the Sicily side and of 51.5 m x 70 m for the Calabria side. Foundation are 7 m high. The excavation area is delimitated by diaphragms walls.

The Terminal Structure is provided with walkways and hatches to allow structural components to be maintained and inspected.

The Terminal Structure is painted in the outer surface for corrosion protection whilst only a single coat of primer is applied to the inner surfaces which are protected by a dehumidification system.

1.6 Piers

The piers are made of reinforced concrete and have a 2-cell shaped section.

A section of the pier is shown in Figure 1.3:

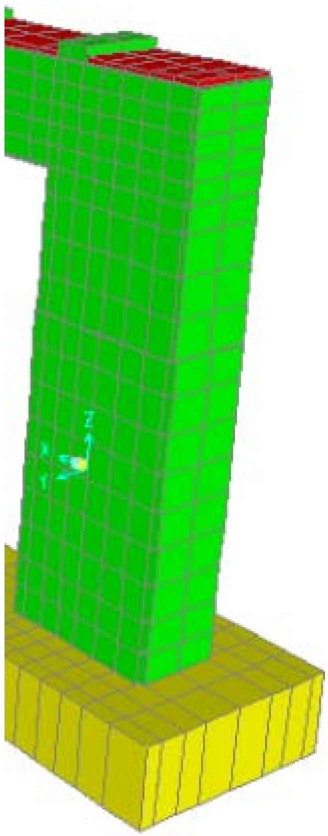


Figure 1-3: View of the pier

For the ULS limit state with EQ the higest utility ratio in flexural bending is 1.126 in the in the Sicilia pier sea side and is this verified satisfactorily as is the shear capacity.

For the ULS limit state with EQ the highest utility ratio in flexural bending is 1.51 in the Calabria pier, land side and is this verified satisfactorily as is the shear capacity.

1.7 Foundation

Foundations are direct on concrete slabs. They comprise an excavation area of 73 m x 76.5 m for the Sicily side and of 51.5 m x 70 m for the Calabria side. Foundation are 7 m high. The excavation area is delimitated by diaphragms walls.

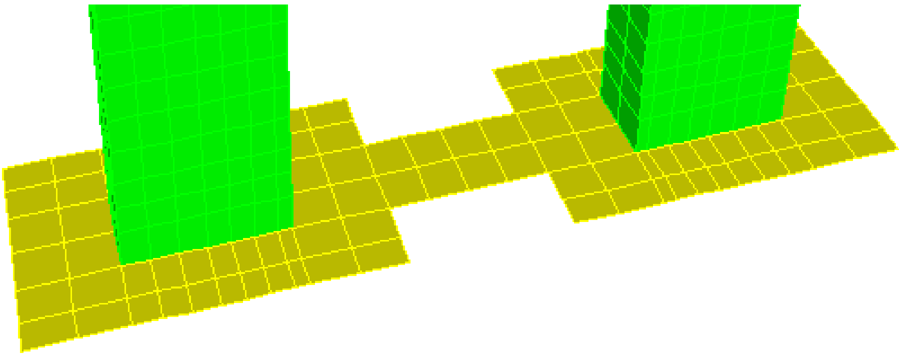


Figure 1-4: Terminal Structures – Foundation slab

Terminal Structure foundation, Sicilia side is verified and the for the ULS limit state with EQ the highest utility ratio in flexural bending is 1.039 in the slab sea side.

Terminal Structure foundation, Calabria side is verified and the for the ULS limit state without EQ the highest utility ratio in flexural bending is 1.05 in the slab sea side.

1.8 Cross beam

Each pier is connected by a reinforced concrete beam which is shaped in a way to allow a zone for jack housing and a zone to position the device which fixes the deck transversally.

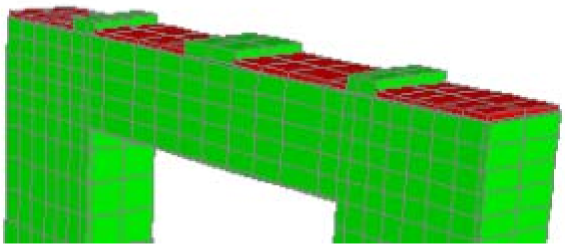


Figure 1-5: Terminal Structures – Cross beam

Terminal Structure substructure, Sicilia side, for the ULS limit state with EQ the highest utility ratio in flexural bending is 1.198 in the cross beam of the pier.

Terminal Structure substructure, Calabria side, for the ULS limit state with EQ the highest utility ratio in flexural bending is 1.32 in the cross beam of the pier.

1.9 Tie-down diaphragm

The tie-down diaphragm is a reinforced concrete beam with following dimensions: 7.0 m x 7.0 m and 2.0 m high.

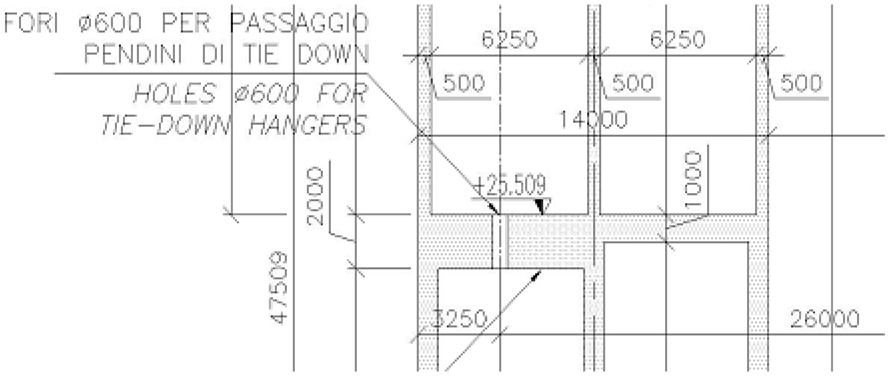


Figure 1-6: Terminal Structures – Tie-down diaphragm

Earthquake loading is entirely governing for the substructure for the Terminal Structures and in the serviceability state the substructure is only stressed lightly, i.e. maximum crack widths are found to be 0.1 mm.

The highest utility ratio in flexural bending is 0.57 in the tie-down diaphragm.

2 Introduction

This report describes the design of the following structural elements of the terminal structures:

- Piers
- Foundations
- Cross beam
- Tie-down support

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 transverse diaphragm walls underneath the foundation have been removed.
- The foundation slab dimensions have changed compared to the tender design (26 m x 70 m maximum for Sicilia side and 21 m x 70 m for Calabria side, excluding the diaphragm walls around the excavated area).
- The pier elevations have been modified accordingly the changing made in the master profiles and in the slopes of the suspended bridge.
- The pier section has been subdivided into two different sections (one with thickness of 0.80 m and the other with of 0.50 m).
- An additional diaphragm adjacent to the tie-down diaphragm has been added, together with a diaphragm in correspondence of the bottom of the slab of the beam connecting the piers.
- The cross beam dimensions and the pier dimension above the diaphragm have been reduced.

The substructures consist of a two piers and a cross beam which are founded on connected concrete slabs. Each pier is 2-cell structure and the cross-beam a single box structure.

2.1 Scope

This report contains the design principles, the modelling description and the design of structural elements of the terminal structure substructure, both Sicily and Calabria side.

2.2 Report Outline

This report is organized into the following sections:

- *Section 1* includes the executive summary, which gives a brief description of the report.
- *Section 2* includes an introduction, provides a list of reference materials, including design specifications, design codes, material specifications, reference drawings and complementary reports.
- *Section 3* provides definitions for terms that are commonly used in referencing particular terminal structure components.

- *Section 4* describes the three limit states that are considered in the terminal structure substructure design, ultimate and structural integrity;
- *Section 5* provides descriptions of the materials that are used for each terminal structure substructure component;
- *Section 6* provides descriptions of the terminal structures substructures design consideration.
- *Section 7* describes the structural analysis methods and the Finite Element Models used to analyze the components of the terminal substructures;
- *Section 8* describes the design criteria, including philosophy and code references, by which the terminal structure superstructure components are verified for the limit states;
- *Section 9* describes the results of FEM analysis and of the design verifications.

2.3 Basic studies

Due to the time gap between signing the contract to the actual start of the work, Italian and European codes and standards have been updated.

2.3.1 Design Basis

New technical standards for construction, NTC08, and new Italian railway codes, RFI DTC-ICI-PO SP INF are now current standard in Italy. These codes are based upon the Eurocodes The following modifications apply to the Basis of Design as defined for the Tender Design:

• QL loads and load combinations are defined according to NTC08 and RFI DTC-ICI-PO SP INF 001 A which is now equal to road and railway traffic loads defined in EN1991-2 (Traffic loads on bridges).

• Geotechnical design is based on requirements in NTC08.

Requirements for fatigue analysis are defined according to NTC08 and RFI DTC-ICI-PO SP INF 003 A (RFI 44F). Only the "safe life" assessment method is applicable for the design of primary load carrying elements.

A general study has been performed to update seismic design criteria for Italy. The results have not been adapted to the Messina Strait Bridge site conditions. Hence the Design Spectra defined for the Tender Design are maintained.

2.4 References

2.4.1 Design Specifications

GCG.F.04.01 “Engineering – Definitive and Detailed Design: Basis of Design and Expected Performance Levels,” Stretto di Messina, 2004 October 27.

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.

CG1000-P-RG-D-P-GE-00-00-00-00-00-02 “Design Basis, Structural, Annex“

CG.10.03-P-CL-D-P-CG-S4-00-00-00-00-01, “Equivalent Stiffness matrices for the Soil-Foundation System”

CG.10.03-P-CL-D-P-CG-S4-00-00-00-00-02, “Equivalent Stiffness and Damping Matrices for the Soil-Foundation System”

2.4.2 Design Codes

NTC-08: DM14.1.2008 - “Norme tecniche per le costruzioni,” 2008 (NTC08).

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

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

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

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

EN 1993 Eurocode 3: Design of Steel Structures – Part 1-10: Selection of steel for fracture toughness and through thickness properties.

EN 1993 Eurocode 3: Design of Steel Structures – Part 1-10: Selection of steel for fracture toughness and through thickness properties.

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

EN 1998 Eurocode 8: Design of structures for earthquake resistance.

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

2.4.3 Material Specifications

EN 10025-1:2004 Hot rolled products of structural steels – Part 1: General technical delivery conditions.

EN 10025-2:2004 Hot rolled products of structural steels – Part 2: Technical delivery conditions for non-alloy structural steels.

EN 10025-3:2004 Hot rolled products of structural steels – Part 3: Technical delivery conditions for normalized / normalized rolled weldable fine grain structural steels.

EN 10025-4:2004 Hot rolled products of structural steels – Part 4: Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels.

EN 10164:1993 Steel products with improved deformation properties perpendicular to the surface of the product – Technical delivery conditions.

EN ISO 898-1:2001 Mechanical properties of fasteners made of carbon steel and alloy steel – Part 1: Bolts, screws and studs (ISO 898-1:1999).

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

UNI EN 14399:2005-3 High-strength structural bolting assemblies for preloading - Part 3: System HR - Hexagon bolt and nut assemblies

EN ISO 14555:1998 Welding-Arc stud welding of metallic materials. May 1995.

EN ISO 13918:1998 Welding-Studs and ceramic ferrules for arc stud welding-January 1997.

2.4.4 Drawings

Terminal Structure - Substructure - Sicily side	
CG1002-P-AX-D-P-ST-F4-VS-00-00-00-03_A	General layout
CG1002-P-AX-D-P-ST-F4-VS-00-00-00-01_B	General Arrangement
CG1002-P-BX-D-P-ST-F4-VS-00-00-00-01_B	Foundation plan and sections
CG1002-P-BX-D-P-ST-F4-VS-00-00-00-02_B	Concrete Dimensions Piers 1/2
CG1002-P-BX-D-P-ST-F4-VS-00-00-00-03_B	Concrete Dimensions Piers 2/2
CG1002-P-PX-D-P-ST-F4-VS-00-00-00-01_B	Reinforcement Foundation Plan 1/2
CG1002-P-PX-D-P-ST-F4-VS-00-00-00-02_A	Reinforcement Foundation Plan 2/2
CG1002-P-WX-D-P-ST-F4-VS-00-00-00-01_B	Reinforcement Foundation and Sections 1/3
CG1002-P-WX-D-P-ST-F4-VS-00-00-00-02_B	Reinforcement Foundation and Sections 2/3
CG1002-P-WX-D-P-ST-F4-VS-00-00-00-06_A	Reinforcement Foundation and Sections 3/3
CG1002-P-WX-D-P-ST-F4-VS-00-00-00-03_B	Reinforcement - Piers, Sections and Details 1
CG1002-P-WX-D-P-ST-F4-VS-00-00-00-04_B	Reinforcement - Piers, Sections and Details 2
CG1002-P-WX-D-P-ST-F4-VS-00-00-00-05_B	Reinforcement - Piers, Sections and Details 3
CG1002-P-AX-D-P-ST-F4-VS-00-00-00-02_B	Temporary work and jet grouting
Terminal Structure - Substructure - Calabria side	
CG1002-P-AX-D-P-ST-F4-VC-00-00-00-03_A	General layout
CG1002-P-AX-D-P-ST-F4-VC-00-00-00-01_B	General Arrangement
CG1002-P-BX-D-P-ST-F4-VC-00-00-00-01_B	Foundation plan and sections
CG1002-P-BX-D-P-ST-F4-VC-00-00-00-02_B	Concrete Dimensions Piers 1/2
CG1002-P-BX-D-P-ST-F4-VC-00-00-00-03_B	Concrete Dimensions Piers 2/2
CG1002-P-PX-D-P-ST-F4-VC-00-00-00-01_B	Reinforcement Foundation Plan 1/2

CG1002-P-WX-D-P-ST-F4-VC-00-00-00-01_B	Reinforcement Foundation and Sections 1/3
CG1002-P-WX-D-P-ST-F4-VC-00-00-00-02_B	Reinforcement Foundation and Sections 2/3
CG1002-P-WX-D-P-ST-F4-VC-00-00-00-06_A	Reinforcement Foundation and Sections 3/3
CG1002-P-WX-D-P-ST-F4-VC-00-00-00-03_B	Reinforcement - Piers, Sections and Details 1
CG1002-P-WX-D-P-ST-F4-VC-00-00-00-04_B	Reinforcement - Piers, Sections and Details 2
CG1002-P-WX-D-P-ST-F4-VC-00-00-00-05_B	Reinforcement - Piers, Sections and Details 3
CG1002-P-AX-D-P-ST-F4-VC-00-00-00-02_B	Temporary work and jet grouting

2.4.5 Complementary Reports

CG1000-P-RG-D-P-SV-00-00-00-00-00-01, "Global IBDAS Model Description".

COWI Document: A9055-NOT-3-001, “QL Road Traffic Loads”, 20. May 2010

COWI Document: A9055-NOT-3-002, “QL Rail Traffic Loads”, 20. May 2010

COWI Document: A9055-NOT-3-003 “QL Load Combinations”, 20. May 2010

CG1000-P-CL-D-P-SS-A0-AP-00-00-00-01, “Design report - Bridge Bearings”

CG1000-P-CL-D-P-SS-A0-AM-00-00-00-01, “Design report - Expansion joints”

CG1000-P-SP-D-P-SS-A0-AP-00-00-00-01, “Performance Specification - Bridge Bearings”

CG1000-P-SP-D-P-SS-A0-AM-00-00-00-01, “Performance Specification - Buffers”

CG1000-P-SP-D-P-SS-A0-AM-00-00-00-02, “Performance Specification-Expansion joints, Railway”

CG1000-P-SP-D-P-SS-A0-AM-00-00-00-03, “Performance Specification - Expansion joints, Roadway”

3 Nomenclature

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

Pier – the vertical elements of the terminal substructure, extending from the top of the concrete foundation slab to the underside of the cross beam.

Cross Beam – the transverse beam connecting the piers and that supports the bearings for the superstructure of the Terminal Structure.

Foundation slab - the direct foundation of the piers.

Diaphragm walls - vertical walls to contain the excavation area for the terminal structure foundation.

Longitudinal Webs – the vertical longitudinal plates inside the Terminal Structure connecting the top and bottom.

Diaphragms – the vertical transverse plates inside the Terminal Structure connecting the facades.

Transverse Truss – the transverse steel truss structure inside the Terminal Structure connecting the facades.

Predalles – precast concrete elements used to support the in-situ cast concrete deck.

Jack housing – Space inside the cross beam to host temporary jacking.

Tie-down – hanger cable connecting to the terminal substructure.

U.R. – utility ratio, i.e. the ratio between demand/capacity.

4 Limit States

This section describes the limit states and corresponding performance requirements governing the proportioning of the tower components, in accordance with the project design basis GCG.F.04.01 and NTC08. The performance of terminal structure components is verified at Serviceability Limit States (1 and 2) Ultimate Limit States and Fatigue Limit States.

4.1 Serviceability Limit States

NTC08 Section 2.2.2 defines the following Serviceability Limit States (SLS) that are to be evaluated in a structural design:

- Local damage that can reduce the durability of the structure.
- Displacement or deformations that could limit the use of the structure, its efficiency and its appearance.
- Displacement or deformations that could compromise the efficiency and appearance of non-structural elements, plants and machinery.
- Vibrations that could compromise the use of the structure.
- Damage caused by fatigue that could compromise durability.
- Corrosion and/or excessive deterioration in materials due to atmospheric exposure.

The project design basis GCG.F.04.01 Section 3.1 specifies the performance requirements for the structure under two levels of serviceability, or normal usage loads. The SLS performance requirements are listed in Table 4-1.

Limit State	***Performance Requirement***
SLS1	*Road and rail runability is guaranteed.* *No structural damage.* *Structure remains elastic and all deformations are reversible.*
SLS2	*As for SLS1 except that only rail runability is guaranteed.*

Table 4-1: SLS performance requirements.

4.2 Ultimate Limit States

NTC08 Section 2.2.1 defines the following Ultimate Limit States (ULS) that are to be evaluated in a structural design:

- Loss of equilibrium of the structure or part of it.
- Excessive displacement or deformation.
- Arrival at the maximum resistance capacity of parts of the structure, joints or foundations.
- Arrival at the maximum resistance capacity of the structure as a whole.
- Arrival at ground collapse mechanisms.

- Failure of frames and joints due to fatigue.
- Failure of frames and joints due to other time-related effects.
- Instability of parts of the structure or structure as a whole.

The project design basis GCG.F.04.01 Section 3.1 specifies the performance requirements for the structure under ultimate or rare loads. The performance requirements are listed in Table 4-2.

Limit State	***Performance Requirement***
ULS	*Temporary loss of serviceability is allowed.* *The main structural system maintains its full integrity.* *Structural damage to secondary components is repairable by means of extraordinary maintenance works.*

Table 4-2: ULS performance requirements.

5 Materials

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

5.1 Concrete and reinforcement

5.1.1 Concrete

All structural concrete to be in accordance with EN 206-1:2001 with following changes and additions:

Concrete Type	**Foundations**	**Piers**
Concrete Grade	**C32/40**	**C40/50**
Time to develop Strength	28 days	28 days
Environmental class	XC4+XS1	XC4+XS1
Consistency Class	S4/S5	S4/S5
Max aggregate size	32 mm	25 mm
Cement	CEM III/B in accordance to EN 197-1	
Max Total Alkali content of cement	0.6%	0.6%

Min. Cement content in Kg/m^3	360	360
Max W/C ratio	0.42	0.42
Chloride content class	0.2	0.2
Max. Alkali content of concrete	3kg equiv. Na_2O per m^{3}of concrete	
Max. Sulfate content of concrete	4% SO_3 by weight of cement	
Concrete composition	Pre-testing of mixes to document compliance with the durability/strength requirements	
Water	From public supply distribution net	
Aggregates	Natural sand, natural gravel, or crushed stone in accordance with EN 12620	
Max Aggregate expansion (Alkali/Silica)	0.10% after 14 days according to ASTM C1260	
Max. Acid-Soluble Sulfate content of aggregates	0.2% according to EN 1744-1	
Admixtures	Admixtures containing chlorides shall not be used	
Max Chloride migration coefficient	4x10^{-2} m^2/s according to NT build 492 after 60 days	
Nominal cover to carbon steel	75	100
Nominal cover to stainless steel	-	50
Early age crack requirement and control	No early age cracks are allowed, temperature/stress analysis to be performed to document that measures for temperature control will ensure crack-free concrete. Input parameters to the analysis to be based on actual documented transient (time/age dependant) concrete properties. Max tensile stress/tensile strength ratio of 0.9.	
Max. concrete temperature during hydration	70°C	
Max heating in adiabatic conditions after 3 days	300kJ/kg cement	
Min. curing period	14 days (alternatively use of curing compound, water retention efficiency index>75% after 72 hours)	
Construction joints	Construction joint shall be cleaned, free of dust and slurry and thoroughly saturated with water. The coarse aggregates shall be made visible down to a depth of 5 to 10 mm	

Table 5-1: Concrete mechanical properties

5.1.2 Reinforcement

Reinforcement bars shall be made of carbon steel grade B450C quality (hot-rolled, ribbed bars of weldable quality and with high ductility), according to EN 10080.

6 Substructure Design Considerations

This section describes the design principles and modelling of the following structural elements of the terminal structures:

- Piers
- Foundations
- Cross beam
- Tie-down diaphragm

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 transverse diaphragm walls underneath the foundation have been removed.
- The foundation slab dimensions have changed compared to the tender design (26 m x 70 m maximum for Sicilia side and 21 m x 70 m for Calabria side, excluding the diaphragm walls around the excavated area).
- The pier elevations have been modified accordingly the changing made in the master profiles and in the slopes of the suspended bridge.
- The pier section has been subdivided into two different sections (one with thickness of 0.80 m and the other with of 0.50 m).
- An additional diaphragm adjacent to the tie-down diaphragm has been added, together with a diaphragm in correspondence of the bottom of the slab of the beam connecting the piers.
- The cross beam dimensions and the pier dimension above the diaphragm have been reduced.

6.1 Terminal structures foundations

The total dimensions of terminal structures foundation area have been confirmed compared to the Tender design. Few modifications have been however introduced. These are described in the following:

- Slabs geometry has been modified. The rectangular section connecting the 2 piers at each side of the terminal structure has been changed into 2 smaller slabs connected by a reinforced concrete beam.

- The use of reinforcement bars ø 46mm has been substituted by the use of bars with a max ø of 32 mm.

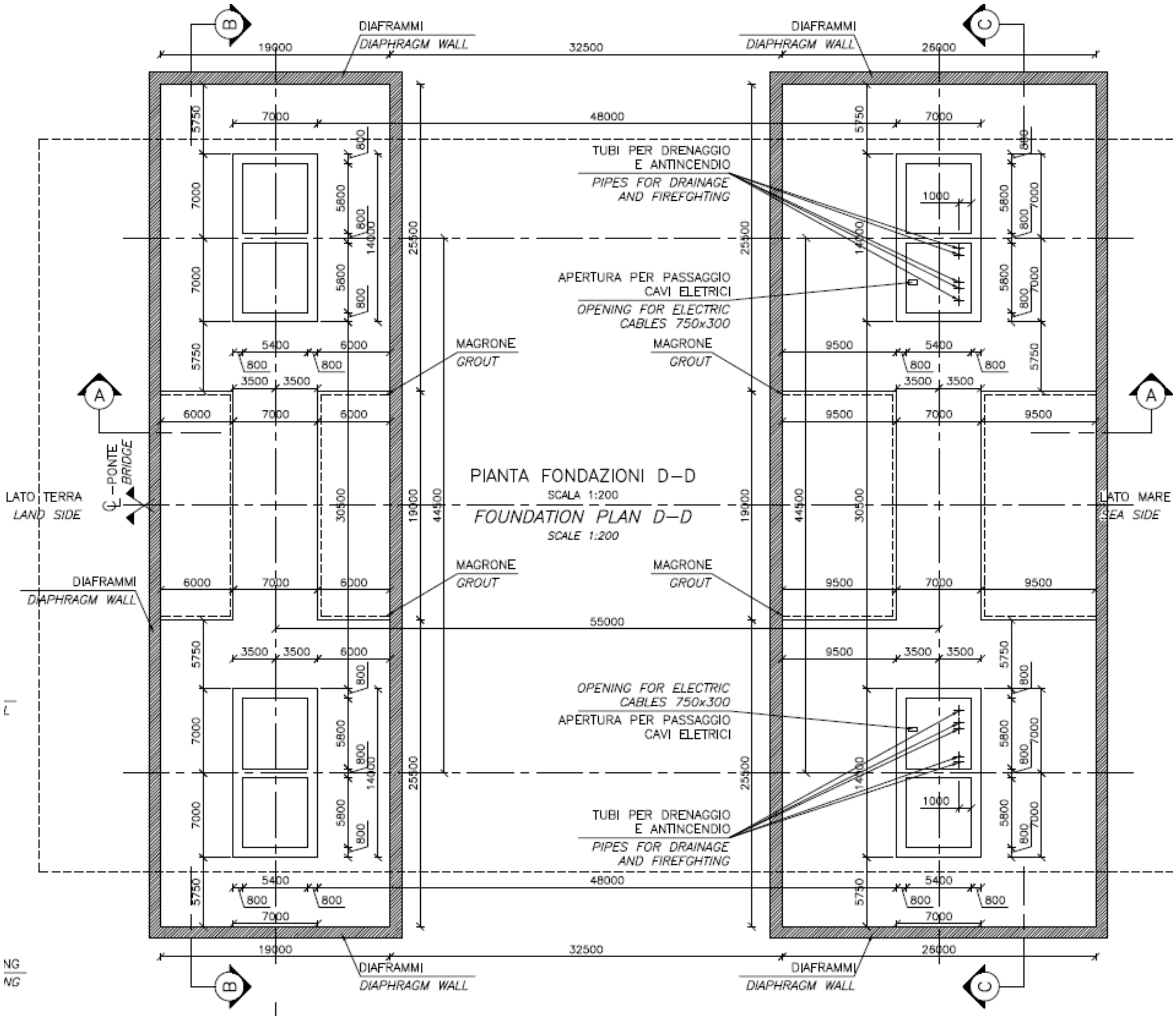


Figure 6.1 Sicilia foundation - Plan

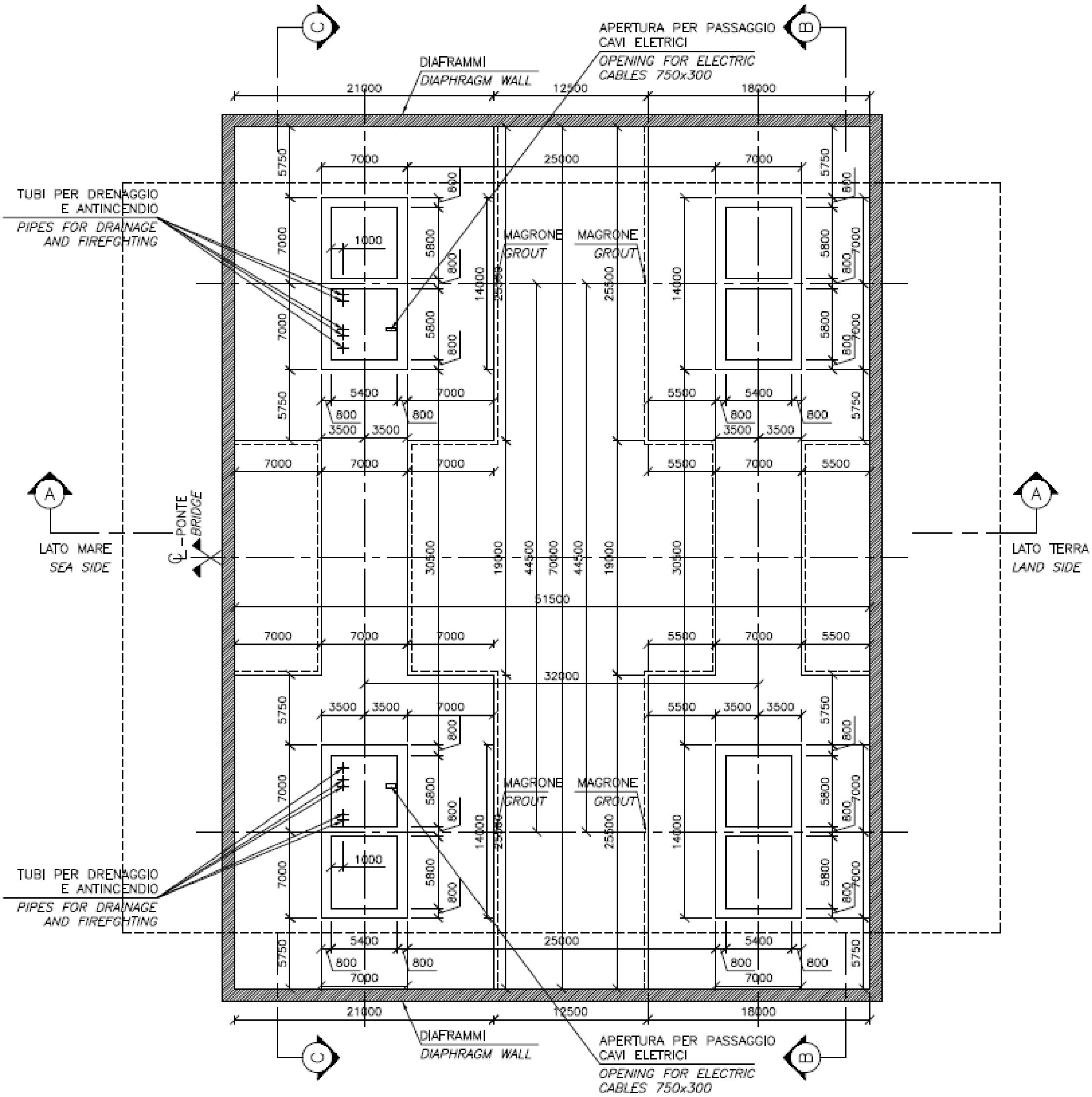


Figure 6.2 Calabria foundation - Plan

6.2 Piers

Terminal structures piers are made, on each side, of 2 legs and 1 transverse beam. The piers are made of reinforced concrete and have a 2-cell shaped section.

Each pier is connected by a reinforced concrete beam which is shaped in a way to allow a zone for jack housing and a zone to position the device which fixes the deck transversally.

The drawings below show the piers elevation in Sicilia and Calabria side respectively:

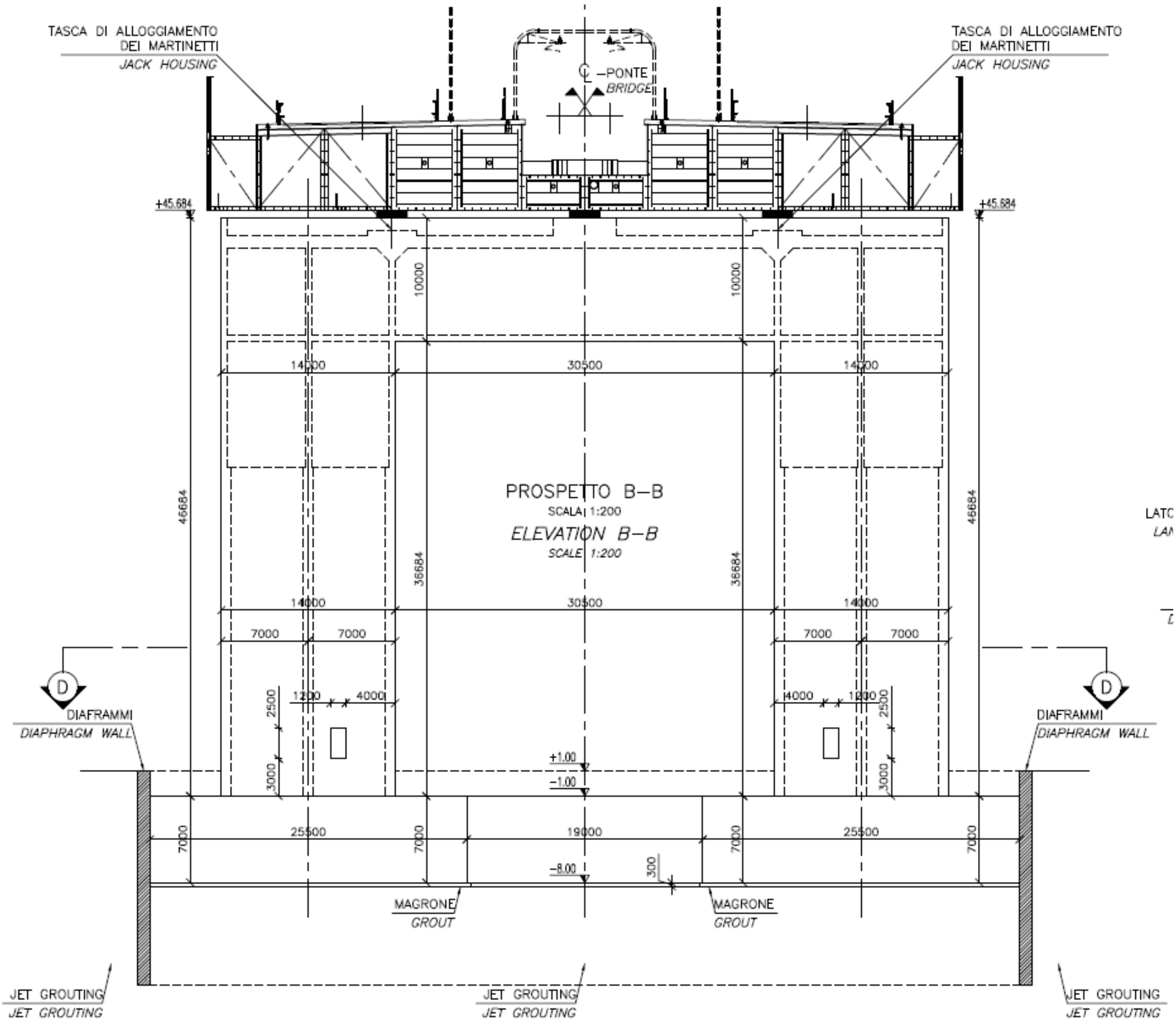


Figure 6.3 Sicilia piers – Elevation

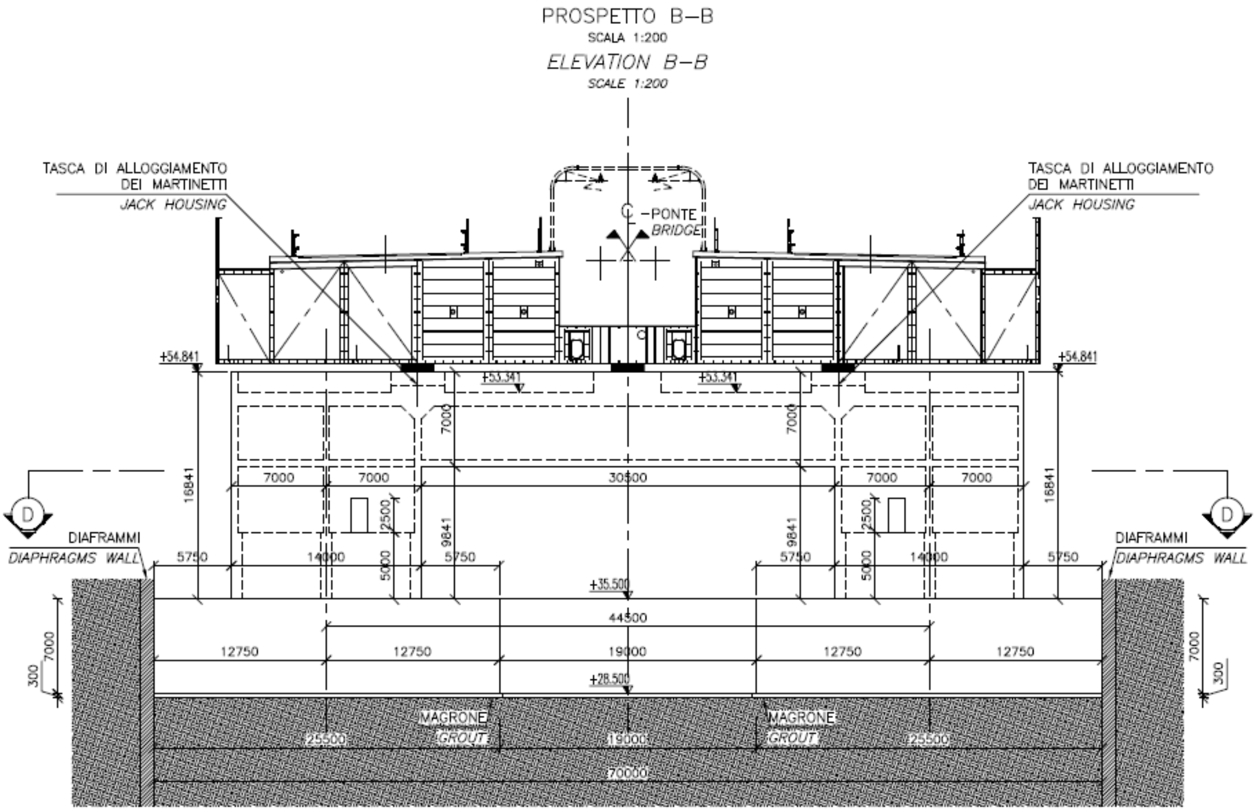


Figure 6.4   Calabria piers – Elevation

Inside the piers the tie-down support is positioned, as shown by the following drawing representing Calabria side:

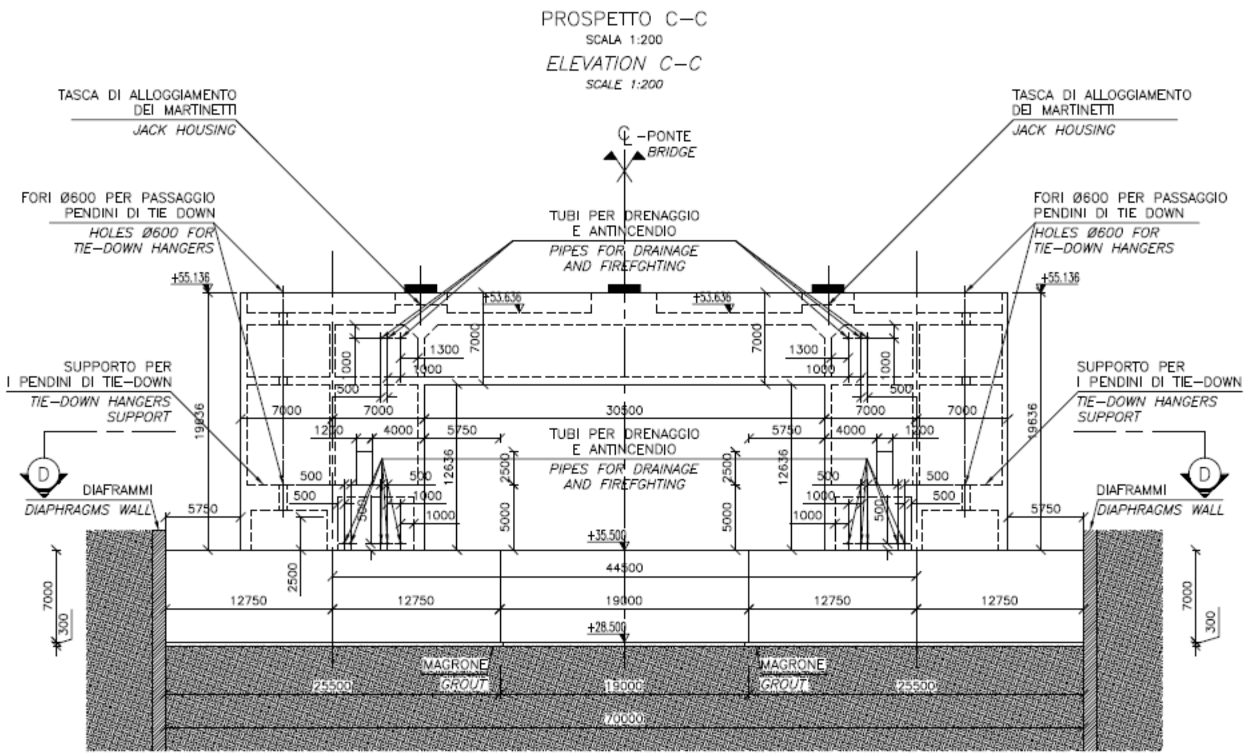


Figure 6.5 Calabria piers – Section

6.3 Tie-down cable and diaphragm

The hanger type will be Parallel Wire Strand (PWS), selected for its improved fatigue performance, increased breaking strength and axial stiffness. PWS results in reduced hanger sizes compared to locked-coil or other systems for a similar wire grade.

Under the Design Basis document the following assumptions are made:

• Hangers are not required to resist SILS loading scenarios.

• Hangers are subject to local QL load combinations at ULS.

Each tie-down cable is connected by sockets to the concrete support inside the piers of terminal structures, as shown in the following figures:

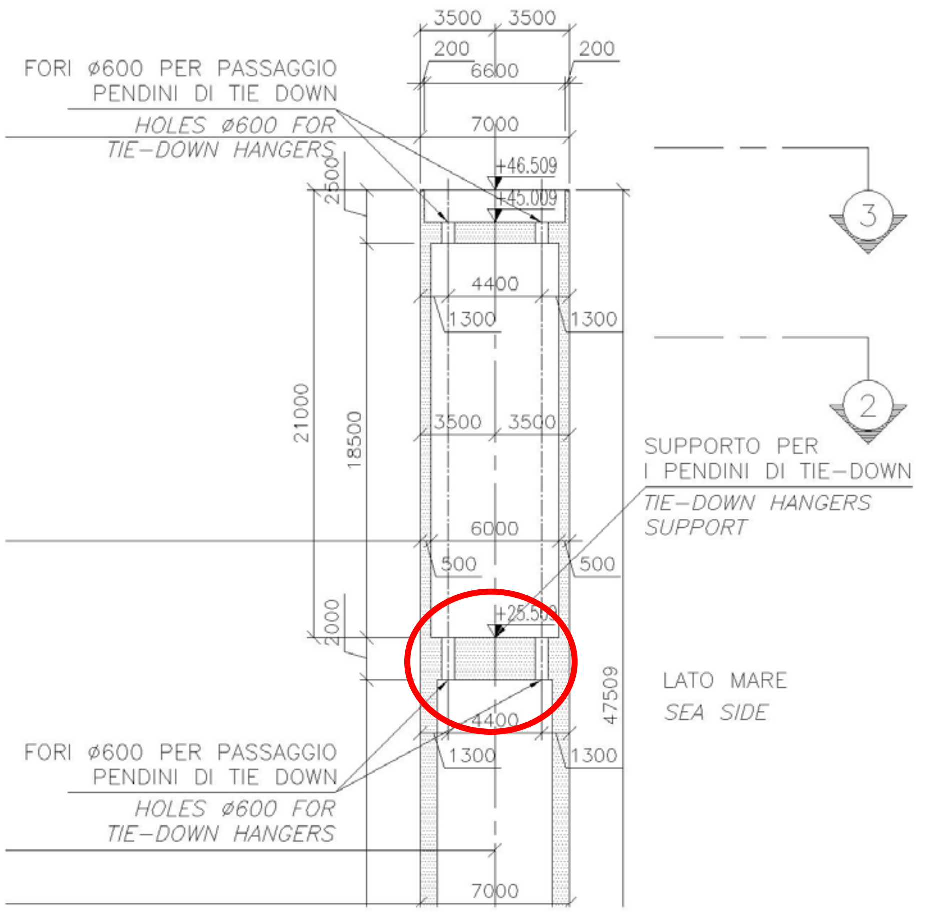


Figure 6.6 Tie-down cables and support

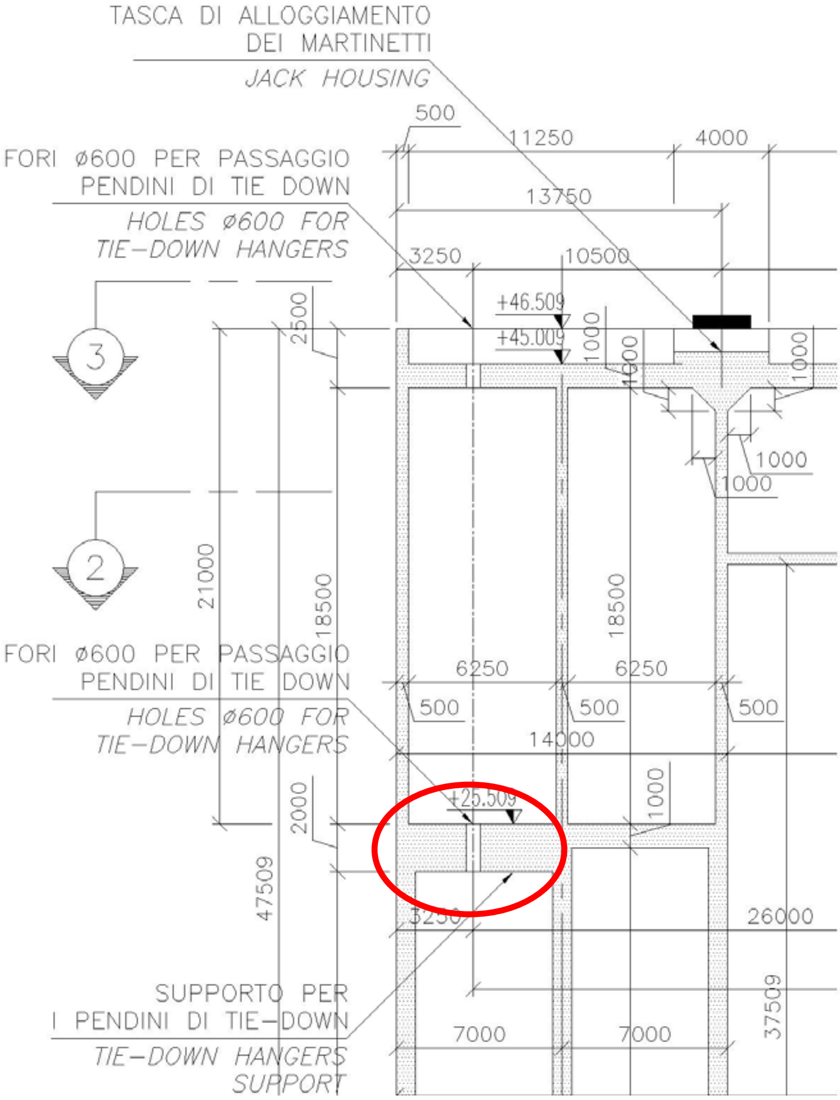


Figure 6.7 Tie-down cables and support

The tie-down support is a reinforced concrete beam with following dimensions: 7.0 m x 7.0 m and 2.0 m high.

A separate Finite Element Model has been performed for this component, using SAP 2000 program.

7 Structural Analysis

The Terminal substructures are modelled and analysed in the developed analysis program SAP 2000 (Structural Analysis Program). This section describes the approach to particular aspects of the structural analysis that affect the terminal substructures design.

SAP 2000 is an integrated computerized system for designing of structures and it returns specialized analyses such:

- Eigenfrequency analysis
- Spectral seismic analysis
- Time history analysis (e.g. for seismic time history analysis)

All calculations performed are based on theory of elasticity and are performed as 1st order analysis.

An SAP 2000 model generally consists of the following 4 model items:

1 Structural Model (geometry model), defining the geometry and materials
2 Finite Element Model (or analysis model)
3 Construction Process Model, defining the construction phases
4 Load Model, defining basic loads and load combinations

The complete terminal structure models and structural analysis, are described in the reports "CG.10.02-P-RX-D-P-SV-S8-VT-000-01-C, Specialist Technical Design Report, Sicily" and "CG.10.02-P-RX-D-P-SV-S8-VC-000-02-C, Specialist Technical Design Report, Calabria."

7.1 Semi- Local FE Model description

This chapter describes the semi-local Finite Element model, build up to study the behavior of the terminal substructures and to design the main elements of the substructure.

The model, better detailed in the next chapters, is based on the following characteristics:

- 2 piers, each consisting of 2 legs and 1 cross girders
- Foundations. The piers foundations geometry has been considered.

7.1.1 Modeling assumptions

Piers foundations have been modeled with shell elements which reproduce their geometry. The insertion plane of the shell is coincident with the medium plane of foundation plinths.

The substructures are made up of 4 legs and 2 transverse beam. These elements represent the reinforced concrete piers, with 2-cell shaped section, of the terminal structures. They have all been modeled with shell elements in the medium plane of the various walls, as shown in the figure below:

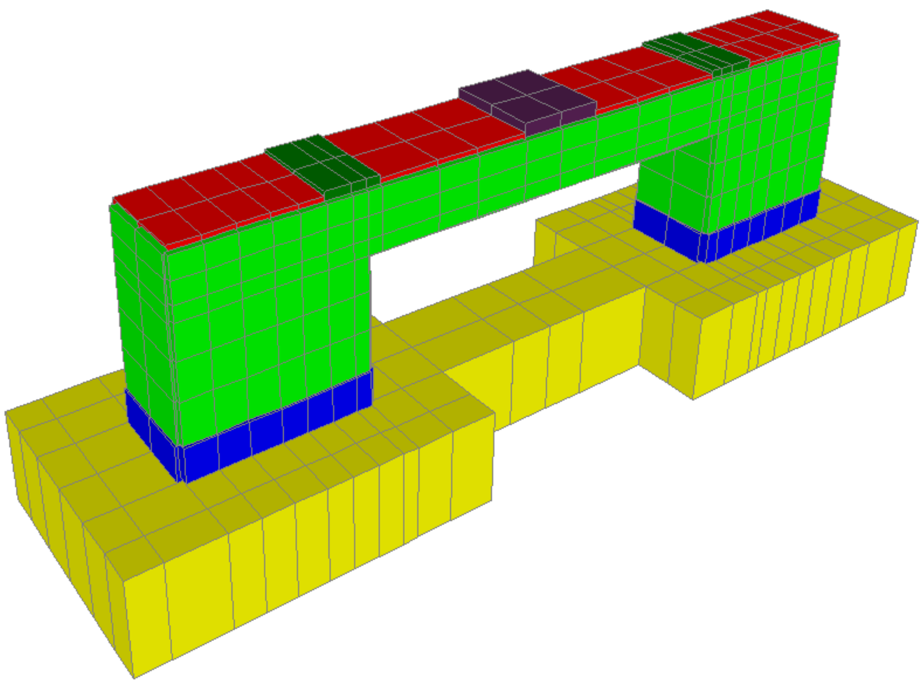


Figure 7.1 Piers view – Calabria side

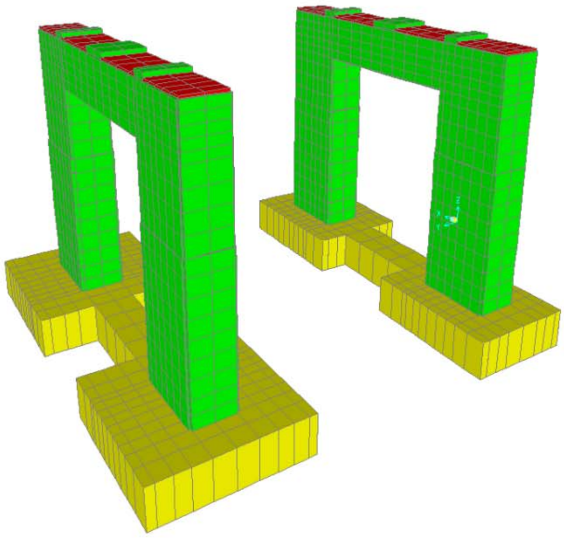


Figure 7.2 Piers view – Sicilia side

7.1.2 Global Reference System

The global reference system is a right hand system. In this system the nodal coordinates and the stresses are defined. In particular: X axis, the first axis, is on the horizontal plane, orthogonal to the bridge axis, Y axis, the second axis, is aligned along the bridge axis and Z axis, the third axis, is vertical and orientated towards the top.

The origin of the coordinate system is positioned along the bridge axis, at the beginning of the terminal structure (connection with the bridge) with z=0 in correspondence of the terrain level.

7.1.3 Material and Basic assumptions

Concrete Piers:	Density:	25.0 kN/m^3
	Strength (cylinder)	40 N/mm^2
	E-modulus:	35,220 N/mm^2
Concrete Foundations:	Density:	25.0 kN/m^3
	Strength (cylinder)	32 N/mm^2
	E-modulus:	33,346 N/mm^2

The coefficient of thermal expansion is taken as:

$$\alpha_{concrete} = 10 \cdot 10^{-6} \text{ per } ^{o}C$$

7.1.4 Element coordinate systems

Element coordinate system is described in the program manual, as reported in "CG.10.02-P-RX-D-P-SV-S8-VT-000-01-C, Specialist Technical Design Report, Sicily" and "CG.10.02-P-RX-D-P-SV-S8-VC-000-02-C, Specialist Technical Design Report, Calabria."

7.2 Stiffness, Masses and Weights

7.2.1 Cross-sectional properties

The detailed modelling of the substructure given by the shell elements is able to reproduce the real inertial and mechanical characteristics of the deck transversal section.

7.2.2 Masses and Weights

In SAP model masses and self weights of primary structural elements are calculated automatically by the program by using the possibility to assign the value of the mass to the materials used in the various sections.

7.3 Boundary and support Conditions

Elements representing the boundary conditions of the terminal substructure can be subdivided into 2 categories:

- Elements representing bearings between superstructure and substructure
- Elements representing soil-foundation interface

7.3.1 Bearings between superstructure and substructure

Bearing devices are modeled with links having characteristics able to realize the given boundary condition, as shown in the scheme in the figure below for Sicily side:

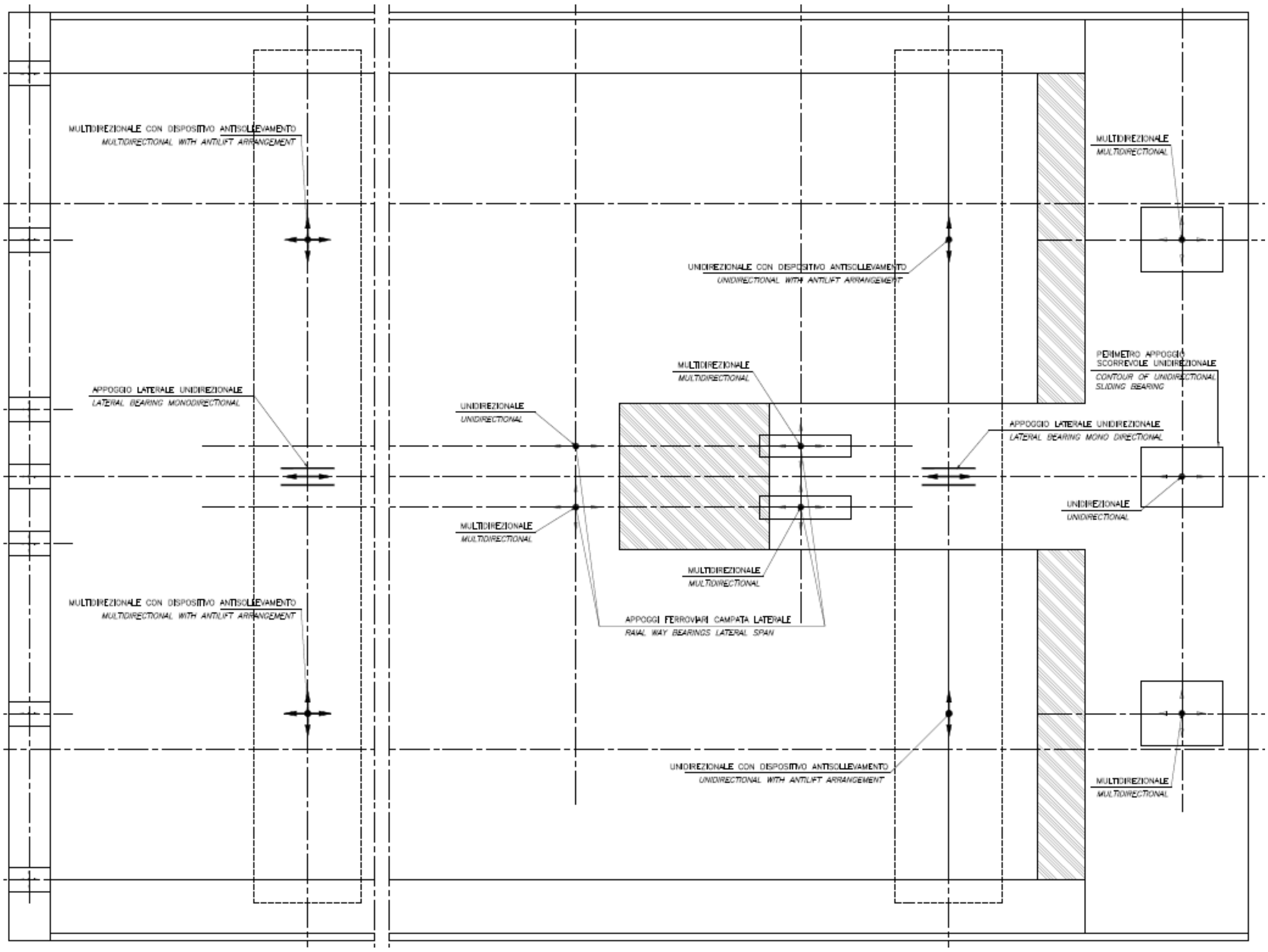


Figure 7.3: Support Conditions: Deck Supports on Piers.

7.3.2 Soil-foundation interface

Soil-foundation interface is modeled applying to linear spring elements to the points representing the nodes of foundation shell elements.

The stiffness of the springs for the static case has been deducted by using the following stiffness matrix:

	X [kN/m]	***Y [kN/m]***	***Z [kN/m]***	***r_X [kNm]***	***r_Y [kN]***	***r_Z [kNm]***
X [kN/m]	2.1 E+07	0	0	0	-1.5 E+08	0
Y [kN/m]	0	2.2 E+07	0	-1.5 E+08	0	0
Z [kN/m]	0	0	1.9 E+07	0	0	0

***r_X* [kNm]**	0	-1.5 E+08	0	9.8 E+10	0	0
***r_Y* [kN]**	-1.5 E+08	0	0	0	1.3 E+11	0
***r_Z* [kNm]**	0	0	0	0	0	1.7 E+11

Table 7-1: Stiffness matrix of soil Sicily side - static

In Calabria side the stiffness of the springs has been deducted by using the following stiffness matrix:

	***X* [kN/m]**	***Y* [kN/m]**	***Z* [kN/m]**	***r_X* [kNm]**	***r_Y* [kN]**	***r_Z* [kNm]**
***X* [kN/m]**	5.3 E+07	0	0	0	1.6 E+08	0
***Y* [kN/m]**	0	5.6 E+07	0	1.7 E+08	0	0
***Z* [kN/m]**	0	0	5.6 E+07	0	0	0
***r_X* [kNm]**	0	1.7E+08	0	3.0 E+10	0	0
***r_Y* [kN]**	1.6 E+08	0	0	0	3.9 E+11	0
***r_Z* [kNm]**	0	0	0	0	0	5.6 E+11

Table 7-2: Stiffness matrix of soil Calabria side - static

The stiffness of the springs for the dynamic case has been deducted by using the following stiffness matrix:

	***X* [kN/m]**	***Y* [kN/m]**	***Z* [kN/m]**	***r_X* [kNm]**	***r_Y* [kN]**	***r_Z* [kNm]**
***X* [kN/m]**	5.0 E+07	0	0	0	-1.7 E+08	0
***Y* [kN/m]**	0	4.8 E+07	0	-1.6 E+08	0	0
***Z* [kN/m]**	0	0	5.8 E+07	0	0	0
***r_X* [kNm]**	0	-1.6 E+08	0	1.4 E+10	0	0
***r_Y* [kN]**	-1.7 E+08	0	0	0	1.8 E+11	0
***r_Z* [kNm]**	0	0	0	0	0	2.7 E+11

Table 7-3: Stiffness matrix of soil Sicily side - dynamic

In Calabria side the stiffness of the springs has been deducted by using the following stiffness matrix:

	X [kN/m]	Y [kN/m]	Z [kN/m]	r_X [kNm]	r_Y [kN]	r_Z [kNm]
X [kN/m]	2.2 E+07	0	0	0	9.6 E+08	0
Y [kN/m]	0	2.1 E+07	0	9.1 E+08	0	0
Z [kN/m]	0	0	3.4 E+07	0	0	0
r_X [kNm]	0	9.1 E+08	0	4.8 E+10	0	0
r_Y [kN]	9.6 E+08	0	0	0	6.1 E+11	0
r_Z [kNm]	0	0	0	0	0	1.0 E+11

Table 7-4: Stiffness matrix of soil Calabria side - dynamic

Reference for these stiffness matrices for soil is made to the following reports:

- CG.10.03-P-CL-D-P-CG-S4-00-00-00-00-01, “Equivalent Stiffness matrices for the Soil-Foundation System”
- CG.10.03-P-CL-D-P-CG-S4-00-00-00-00-02, “Equivalent Stiffness and Damping Matrices for the Soil-Foundation System”

7.4 Loads and Load Combinations

The SAP 2000 model operates with the following types of loads:

- Basic loads defined as 1.0 times the characteristic load.
- Simple load combinations
- Complex load combinations
- Fixed loads and load combinations

Loads are generally defined in the global (right hand) coordinate system.

The basic loads defined are:

- Permanent Loads (Structural weight PP, non-structural components PN)
- Variable man-generated actions (QL)
- Wind Loads (static and dynamic) (VV)

- Temperature Loads (VT)
- Seismic Loads (VS)

The Permanent Loads include all gravity loads such as dead load, super imposed dead load (deck surfacing and "other loads").

Dead load is calculated automatically by SAP 2000 based on the geometry model (and loads defined and included in the construction process model). Weight of the basic materials can be seen in section 6.2.3 of this report.

7.4.1 Permanent load (PP and PN)

Permanent and semi permanent actions are divided into structural self weight (PP) and non-structural self weight (PN), as detailed below:

- **PP**: the structural weight is calculated for all structural components. Increments of weights due to elements which are not present in the model (such as coating and connections) are taken into account using a 10% incremental factor.
- **PN**: the weight of non-structural components includes the weight of any road surfacing, railway trackbed, protections, parapets and wind shields, technological equipment and services, which must be guaranteed along the crossing; scenarios in which any of these weights are removed during the service life of the structure, for the purposes of routine and extraordinary maintenance, has been considered.

On the FE model, in addition to the above mentioned loads, permanent loads, resulting from the Messina main bridge and from the Pantano viaduct, have been inserted in correspondence of the nodes at the bearing positions.
The forces at the interface bridge-terminal structures have been calculated with the global IBDAS model and have been applied as nodal forces.

7.4.2 Variable man-generated actions (QL)

Variable man-generated actions for local sizing (QL) have been implemented according to the following documents:

1) Doc. No. A9055-NOT-3-002, “QL Rail Traffic Loads”
2) Doc. No. A9055-NOT-3-001, “QL Road Traffic Loads”

The highway and railway loads have been disposed along the structure in such a way as to produce the most adverse load effect. Once the design sections have been identified, the position of the load which induces the maximum stresses in the sections is determinate using the influence lines calculated on the global IBDAS model of the bridge.

Once the positions of the rail loads and road loads maximising the stresses in the design section have been individuated, the corresponding actions are reported on the model of the terminal structures, in two different ways:

- Loads are applied on the bearings at the interface between bridge and terminal superstructure and at the interface betweeen Pantano viaduct and terminal supsserstructure, calculated with the IBDAS model for the particualar load case.
- A part of the running loads is applied on the terminal structure itself (depending on the maximising position).

Applied loads are here below described.

7.4.2.1 Road load

Road loads have been applied accordingly to NTC 08 requirements.
The carriageway area includes all areas that can be incorporated permanently or temporarily for use by vehicular traffic. Thereby the total carriageway area of one road deck of 11.95 m2/m has been loaded. 3 load lanes, 3 m wide have been considered together with a remaining lane of 2.95 m for each carriage way.
2 load schemes have been considered:

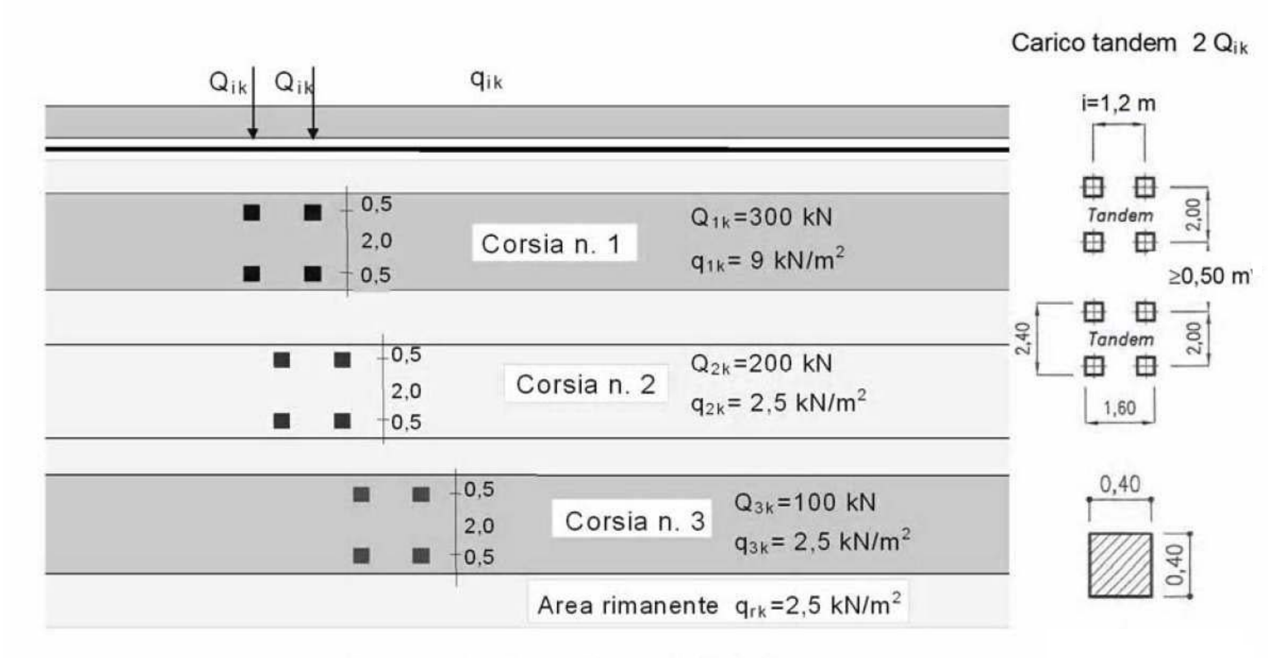
The load arrangement defined in the following table (load scheme 1) has been applied on both carriageways. The dynamic factor is included in the characteristic lane loads.

The load length is limited to maximum 300m.

Location	Tandem system - TS	UDL system
	Axle load Qik (kN)	qik (kN/m^2)
Notional lane number 1	300	9
Notional lane number 2	200	2.5
Notional lane number 3	100	2.5
Remaining area	0	2.5

Table: Load model 1

In the following figure the application of Load scheme 1 is illustrated:



Schema di carico 1 *(dimensioni in [m])*

Figure 7-4: Application of Load model 1

Load model 2 consists of a single axle load of: Qk = 400kN

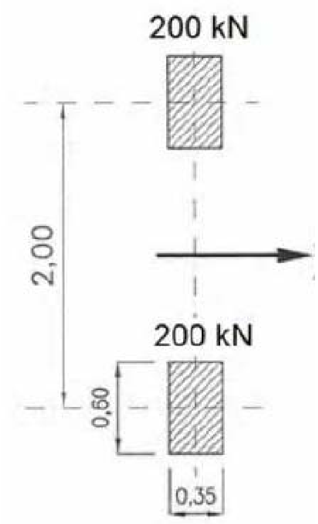


Figure 7-5: Application of model 2 (schema di carico 2)

Once the alignments of shells representing the lanes have been identified, the tandem system loads have been applied diffusing them uniformely on the shell in correspondence on the shell itself.

To the above mentioned loads the following loads longitudinal braking or acceleration force shall be applied in lane number 1 are added as follows:

$Q_{Lk} = 360 + 2.7 \cdot L$

$180 \text{ kN} \leq QLk \leq 900 \text{ kN}$

where L is the actual loaded length under consideration.

The force is assumed to act in the longitudinal direction of the carriageway, parallel with and at the level of the carriageway surface. The force is uniformly distributed over the loaded length.

7.4.2.2 Rail load

In the model 2 shell alignments representing the train binary have been identified and the train loads represented by the load models LM71, SW/0 or SW/ have been applied on these shells.

Load model 71 represents the static effect of vertical loading due to normal rail traffic.

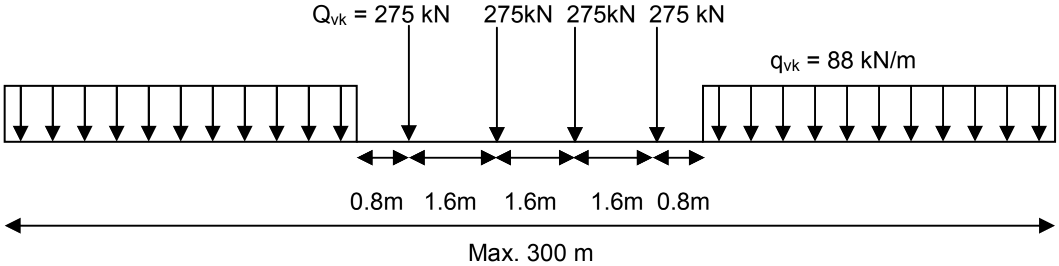


Figure 7-6: Load model 71 with α=1.10

Load model 71 loading has been applied to one or both tracks simultaneously.

The load has been placed in the most adverse position for the element and the uniform part of the LM71 can be applied segmented on the girder in order to increase the load effects of the element under design. α=1.10 for LM71.

Load model SW/0 represents the static effect of vertical loading due to normal rail traffic and SW/2 the static effect of vertical loading due to heavy rail traffic.

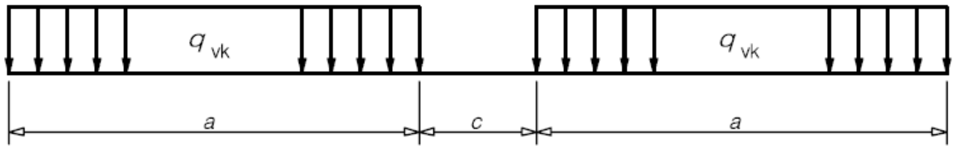


Figure 7-7: Load model SW/0 and SW/2

Load Model	α·qvk [kN/m]	a [m]	c [m]
SW/0	146	15.0	5.3
SW/2	150	25.0	7.0

Load model SW/0 and SW/2 including α=1.10 for SW/0 and α=1.00 for SW/2.

Within a track only one SW/0 or SW/2 shall be applied.

In the FEM model only loads models LM71 e SW/2 have been inserted as the load model SW/0 has been considered not governing and not dimensioning for assessment purposes on the deck elements.

A simply supported beam model of the terminal structure, Sicily side, has been used to evaluate the forces induced by the 3 train load models (LM71, SW/2 and SW/0). It has been found that SW/0 gives smaller forces, compared to LM71 and SW/2.

The results of the above described analyses are here reported:

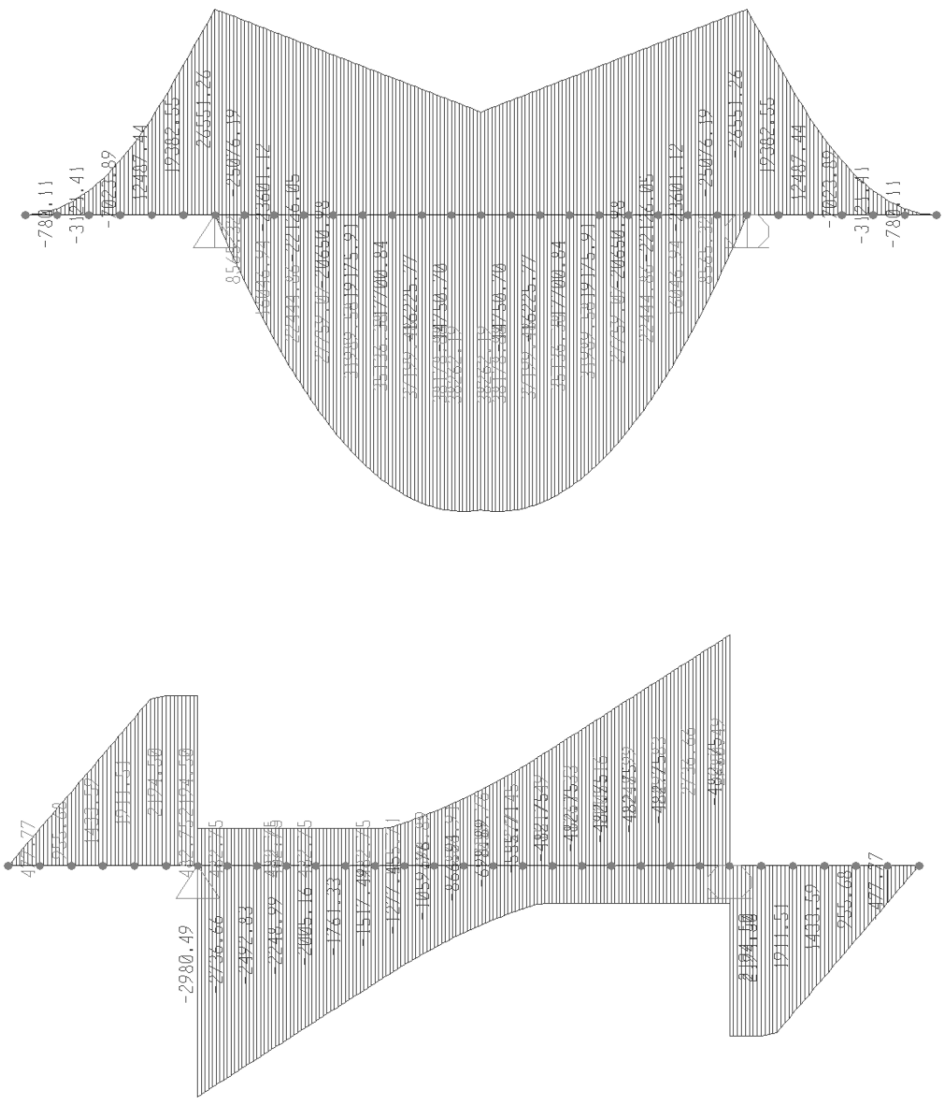


Figure 7-8: Forces induced by Load model SW/0

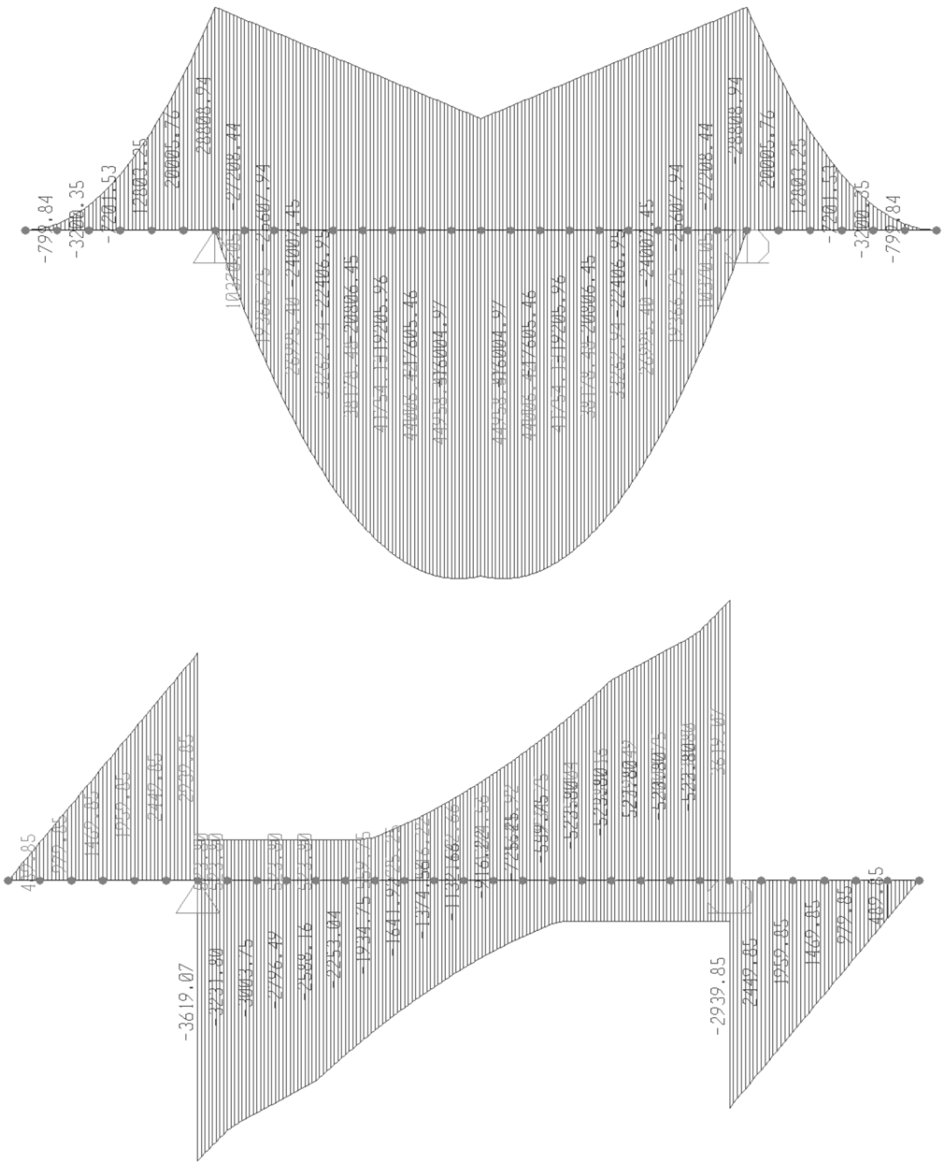


Figure 7-9: Forces induced by Load model SW/2

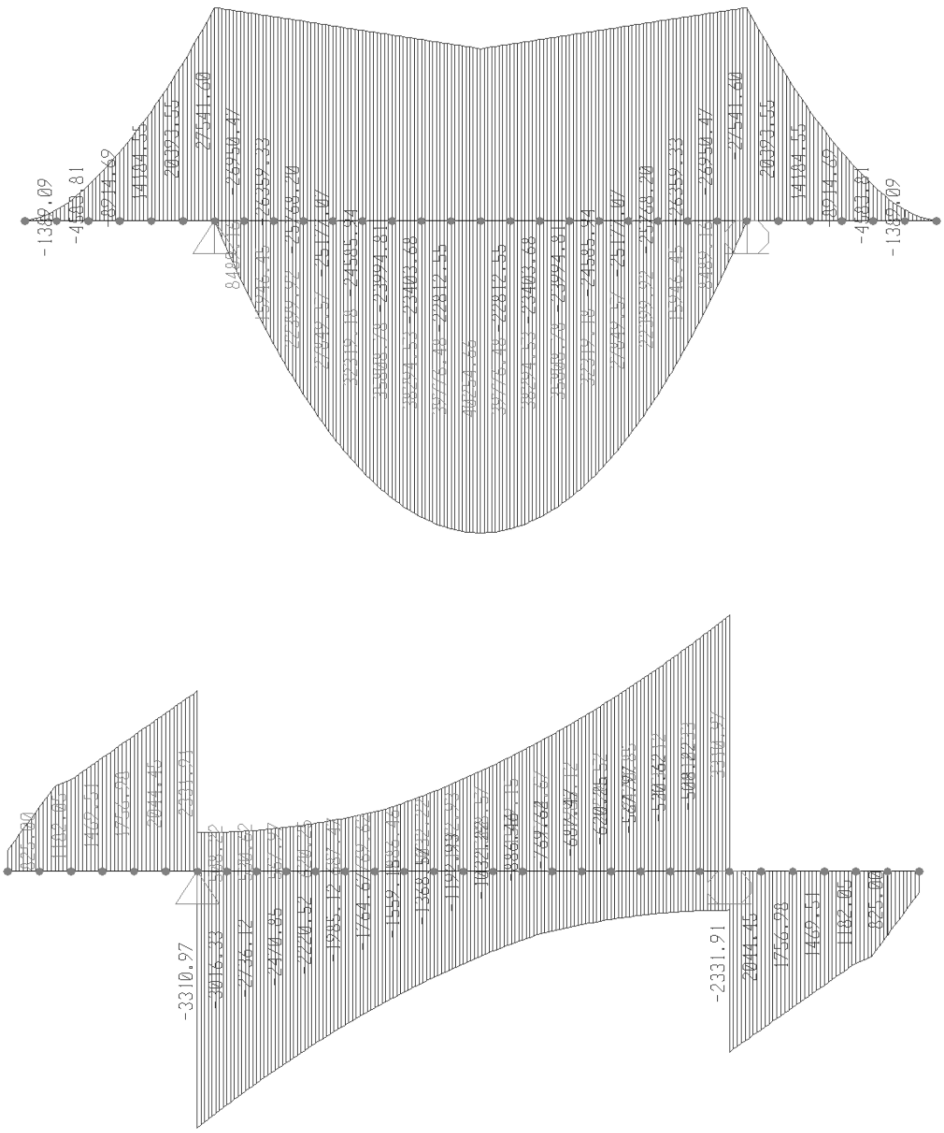


Figure 7-10: Forces induced by Load model LM71

Resulting forces from the train load modela have been amplified with dynamic coefficient Φ = 1.06, calculated with dynamic analyses performer in the global IBDAS mdoel.

Traction and braking forces have been applied at top of rails in longitudinal direction, uniformly distributed over the influence length Lab. The traction and braking forces have been combined with the corresponding verticals loads. Braking in one track has been combined with simultaneous traction in the other track.

It is assumed that the two tracks have permitted direction of travel in opposite directions, i.e. braking or traction in both tracks can not occur.

Traction force is:

Q_{lak} = 33 [kN/m] L [m] ≤ 1000 [kN]

for Load Model 71 and SW/2

Braking force is:

Q_{lbk} = 20 [kN/m] L [m] ≤ 6000 [kN]

for Load Model 71

Q_{lbk} = 35 [kN/m] L [m]

for load model SW/2

Where:

L is the influence length in m of the load effects of the element considered.

The traction and braking loads specified above shall be multiplied by a factor:

α = 1.1 for LM 71

α = 1.0 for SW/2

Nosing force has been applied at top of rails in transversal direction.

Q_{sk} = 100 kN acting horizontally, at the top of the rails, perpendicular to the centre-line of the track. The nosing force specified above has been multiplied by the α factor.

7.4.3 Wind Loading

Wind loading is implemented according to the Design Criteria GCG.F.04.01, section 5.3.

7.4.3.1 Static wind

The static mean wind is implemented with a vertical profile as described in the Design Criteria GCG.F.04.01.

To insert in the model a variation of mean wind speed along the vertical as indicated in the following formula:

$$\bar{u}(z) = \alpha_d \cdot \alpha_r \cdot \left[u_{ref} \cdot k_r \cdot \ln(z/z_0) + 0.01 \cdot z \right]$$

a joint pattern that approximates it with 2 lines has been defined, as shown in the following graph.

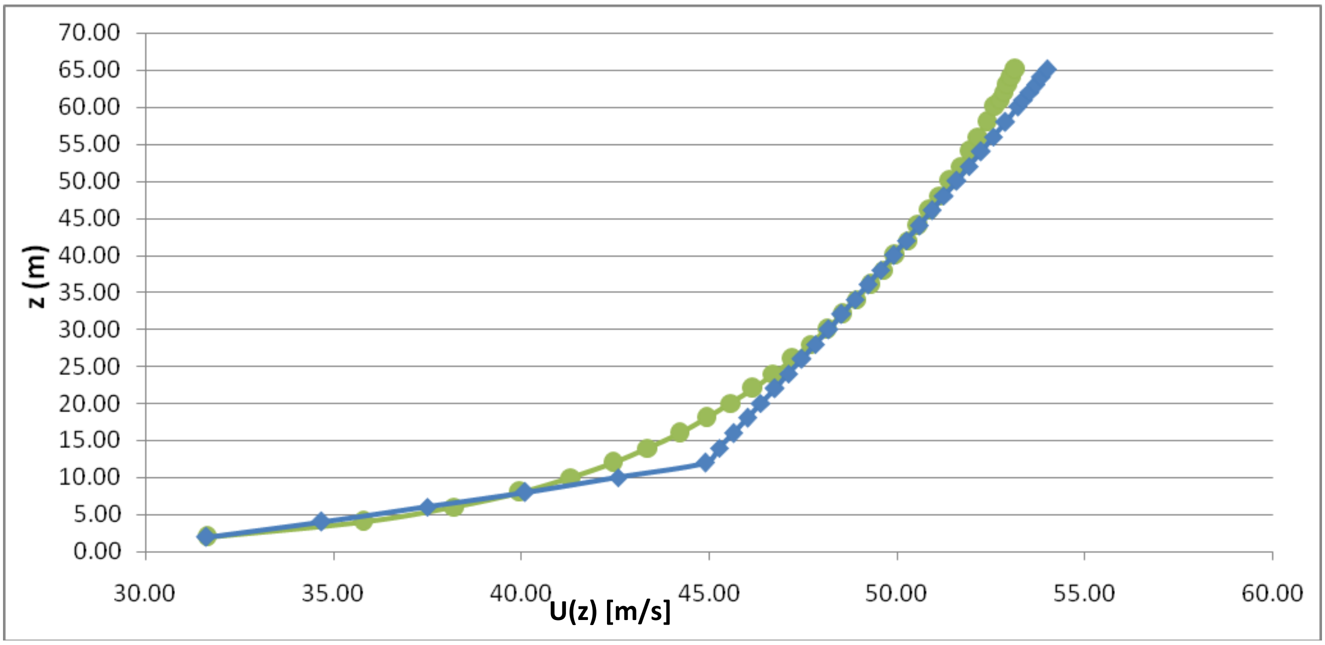


Figure 7-11: Static wind vs elevation

In the SAP model, the 1st linear variation of wind along the vertical is applied up to 12 m elevation and the 2nd variation above this limit.

7.4.4 Temperature Loads

Temperature Loads are implemented according to the Design Criteria GCG.F.04.01, section 5.3.

The coefficients of thermal expansion are defined in section 6.2.3 of this report.Temperature variation are introduced in the model adding I joint patterns able to reproduce along the shells that variation.

7.4.5 Seismic Loads

Seismic load is implemented according to the Design Criteria GCG.F.04.01, section 5.3.2.

The principles used for the seismic analysis are described in the report: CG1000-P-RG-D-P-SV-00-00-00-00-00-01, "Global IBDAS Model Description".

Complete Quadratic Combination (CQC) is used for combining the effects from different modes.

On the semi-local model of the terminal structure the seismic analyses have not been implemented: the stresses due to the seismic actions on the designing sections of the structure have been calculated with the FE global SAP model.

7.5 Load Combinations

The deck components are verified for the load combinations listed in the design basis (GCG.F.04.01) Section 6.8. The load components considered in the load combination tables are defined in Table 2.9. The combinations and associated partial factors for each load component are presented in Table 2.10 and Table 2.11, respectively for SLS and ULS. The partial factor μ can take the value of 0.95 or 1.15 for steel components or 0.95 or 1.25 for concrete components, depending on where the dead load causes a relieving or adverse effect. A hyphen in a cell under a load component column indicates that the load component is not included in the combination represented by the row.

The so defined load combinations, the numbering and the factors applied are reported in the following reports, in Appendix:

- CG1002-P-CL-D-P-ST-S6-00-00-00-00-01_C_Design Report Sicilia Terminal Foundation
- CG1002-P-CL-D-P-ST-S6-00-00-00-00-02_C_Design Report Calabria Terminal Foundation

7.6 Modal Analysis

Modal analysis of the terminal structure is not part of this report. Dynamic behaviour of the structure has been analyzed by COWI with IBDAS global model. Reference is made to the report CG1000-P-RG-D-P-SV-00-00-00-00-00-01, "Global IBDAS Model Description".

7.7 Seismic Analysis

7.7.1 Response Spectrum Analysis

Seismic analysis of the terminal substructure has been performed with IBDAS global model.

For the seismic analysis reference is made to the report CG1000-P-RG-D-P-SV-00-00-00-00-00-01, "Global IBDAS Model Description".

Seismic forces acting in the terminal structures have been selected directly from the IBDAS global model, for load combination which could maximize the stresses in the design chosen sections.

The forces resulting from the IBDAS model were divided into:

- 4 piers for Sicily side (2 piers sea side and 2 piers land side) and 4 piers Calabria side (2 piers sea side and 2 piers land side)
- 2 foundation slabs for Sicily side (1 foundation slab under piers sea side and 1 under the piers land side) and 2 for Calabria side (1 foundation slab under piers sea side and 1 under the piers land side).

Resulting forces in the piers, coming from IBDAS model, have been directly used to verify the sections of the piers of terminal structures in the chosen design location.

Resulting forces on the foundation have not been used directly as the reaction forces were given in a single point at the intersection of 2 rigid elements connecting the piers modeled in the IBDAS global model.

A local FEM in SAP 2000 has been created to model the foundation slab. Forces resulting from the IBDAS model have been applied to the local model.

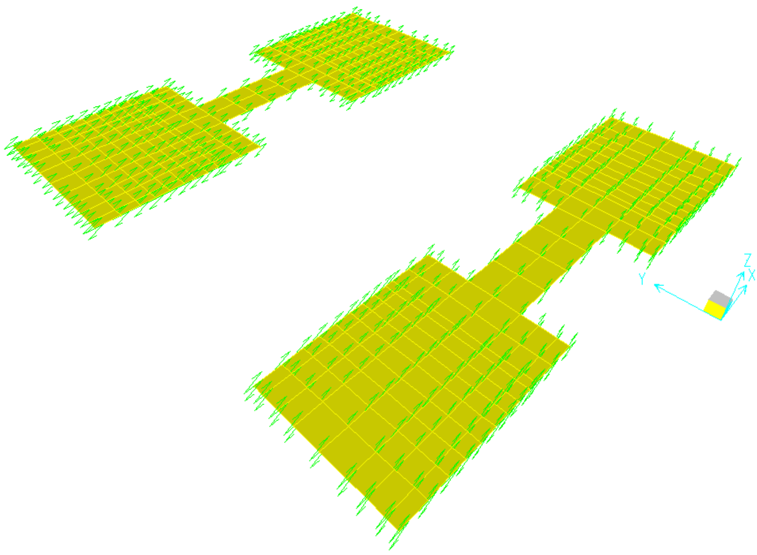


Figure 7-12: Foundation Local Model

Dynamic springs have been used to model the behavior of the ground. The stiffness of the springs for the dynamic case has been deducted by using the following stiffness matrix:

	X [kN/m]	Y [kN/m]	Z [kN/m]	r_X [kNm]	r_Y [kN]	r_Z [kNm]
X [kN/m]	5.0 E+07	0	0	0	-1.7 E+08	0
Y [kN/m]	0	4.8 E+07	0	-1.6 E+08	0	0
Z [kN/m]	0	0	5.8 E+07	0	0	0
r_X [kNm]	0	-1.6 E+08	0	1.4 E+10	0	0
r_Y [kN]	-1.7 E+08	0	0	0	1.8 E+11	0
r_Z [kNm]	0	0	0	0	0	2.7 E+11

Table 7-5: Stiffness matrix of soil Sicily side - dynamic

In Calabria side the stiffness of the springs has been deducted by using the following stiffness matrix:

	X [kN/m]	Y [kN/m]	Z [kN/m]	r_X [kNm]	r_Y [kN]	r_Z [kNm]
X [kN/m]	2.2 E+07	0	0	0	9.6 E+08	0
Y [kN/m]	0	2.1 E+07	0	9.1 E+08	0	0
Z [kN/m]	0	0	3.4 E+07	0	0	0
r_X [kNm]	0	9.1 E+08	0	4.8 E+10	0	0
r_Y [kN]	9.6 E+08	0	0	0	6.1 E+11	0
r_Z [kNm]	0	0	0	0	0	1.0 E+11

Table 7-6: Stiffness matrix of soil Calabria side - dynamic

7.8 Results from the calculation

In the calculation, in order to verify the structural elements, transversal sections (section cuts) have been defined. They are made up of groups of shells and joints and are the sections where the program calculates directly the stresses needed in the verification.

By default the positive local 1, 2 and 3 axes of the section cut correspond to the global X, Y and Z axis, respectively.

Section cut forces are reported at a single point in the local coordinate system defined for the section cut. Six different force components are reported at that single point.

They are:

- **F1:** A force in the section cut local 1-axis direction.
- **F2:** A force in the section cut local 2-axis direction.
- **F3:** A force in the section cut local 3-axis direction.
- **M1:** A moment about the section cut local 1-axis.
- **M2:** A moment about the section cut local 2-axis.
- **M3:** A moment about the section cut local 3-axis.

Section cut forces are reported as forces acting on the objects that make up the group that defines the section cut. An example of this is described below. Positive section forces act in the same direction as the positive section cut local axis. The sense of positive moments can be determined using the right-hand rule.

7.9 Tie down-Hanger Support

7.9.1 FE Modelling Description

The model used to determinate the stresses on the support beam of tie-down cable is made entirely by shell elements. The model has been implemented with the software SAP 200 from Computers and Structures.

The support has been modeled with a plate fixed on each side, as shown in the following drawing:

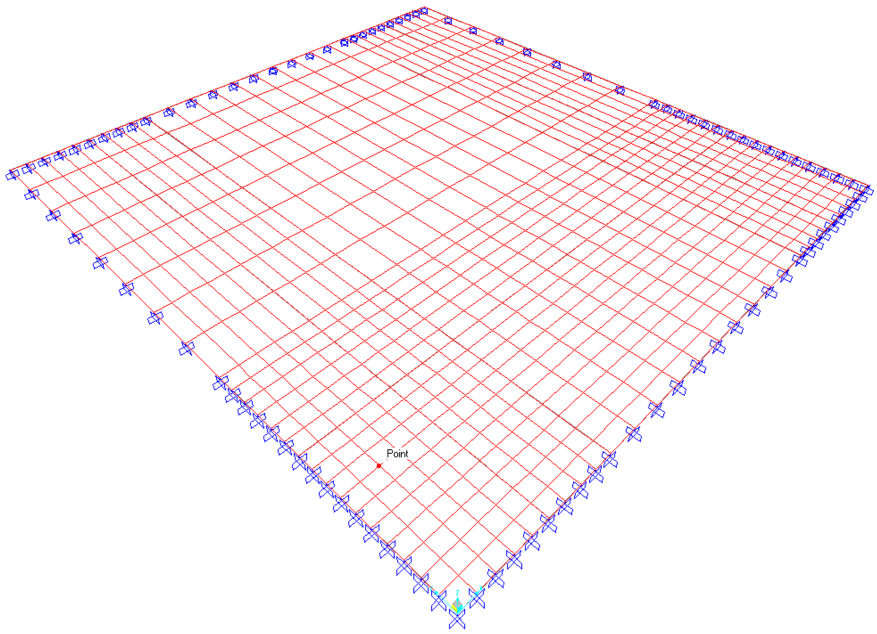


Figure 7.13 Model for tie-down support

7.10 Loads and combinations

The self weight of the support beam has been calculated by the software, assigning the exact section thickness value to the shell elements.

The loads given from the tie-down to the support have been applied accordingly to the following scheme load:

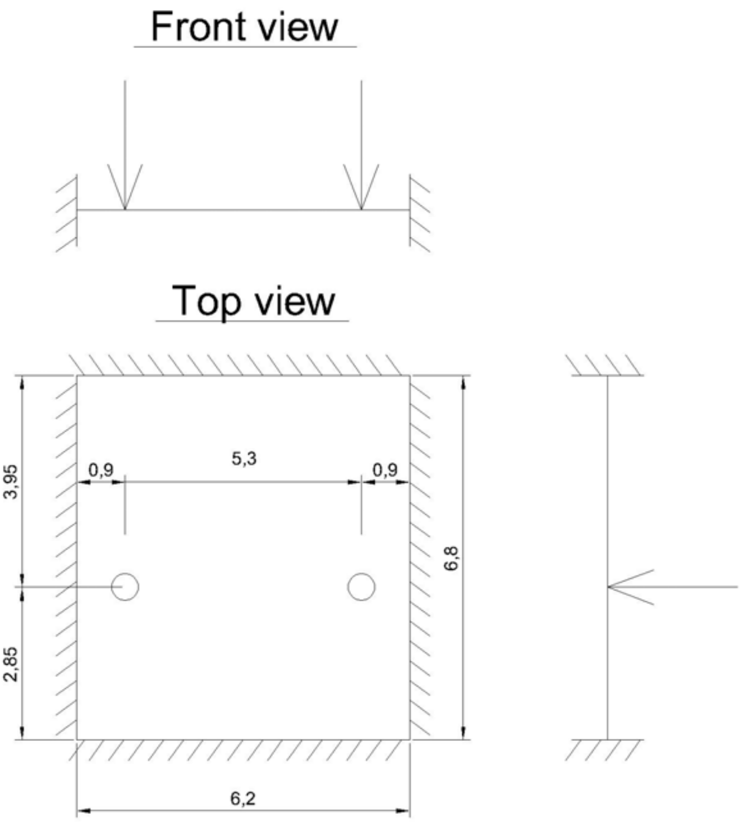


Figure 7.14 Scheme load for tie-down support

The loads have been combined as required by the Design Criteria GCG.F.04.01.

For more details reference is made to documents CG1002-PCLDP-STS6-00000000-01_C and CG1002-PCLDP-STS6-00000000-02_C.

8 Design Verifications

8.1 Verification Basis

The verification are developed according to the guidelines stated in the NTC08. The calculations are based on sectional forces from SAP FE- model.

The program used to verify the concrete sections is GEOSTRU.

8.1.1 Program Input - Output

The verications presented in the following are carried out by use of the commercial program GEOSTRU and the verifications contain the following steps.

Input

Sectional forces for the combinations for ULS and SLS (Char., Freq, and Quasi-perm.) are tabelled. The forces have been taken out so to maximise either the axial force (F3), the moments (M1 or M2) and the shear forces (F1 or F2).

The tabelled forces are derived from the SAP 2000 model.

For the verification material parameters are given and for SLS also allowable crack widths are given according to whether it is a frequent or quasi-permanent load combination.

The section to be verified is geometrically defined and reinforcement is also defined. Note, in some instances a bar diameter Ø45 mm is applied which is not the actual bar diameter but it has been applied as equivalent to 2 smaller bars.

The initially listed sectional forces are fed into the program which then verifies the sections in relation to the applied sectional forces.

Output

Verifications are given in relation to bending and axial load, in relation to shear and finally in relation crack widths.

The verification is confirmed with S (Si = yes) or N (No = no) and the confirmation is listed under the column "Ver".

Axial load - Flxural Bending

For the axial load - flexural bending interaction is also given the ratio of capacity vs. demand under the column "Mis. Sic" (Misura Sicurezza = safety level) - see example is given below.

S.Comb.	Ver	N	Mx	My	N ult	Mx ult	My ult	Mis.Sic.
1	S	16513087	0	-21887379	16513092	0	-356535128	16.290

Example: Mis. Sic. = capacity / demand = My ult / My = -356535128 / -21887379 = 16.290

For the axial load - flexural bending verification, the axial capacity "Nult" is set app. equal to the the applied axial load to find the remaining flexural bending capacity.

Shear

Shear is verified by comparing the applied load to the minimum of the individual capacity of either the concrete (Vcd) or the reinforcement (Vvd).

Crackwidth

Cracking is verified by comparing the calculated crackwidth (Ap.Fess.) with the allowable crackwidth.

When calculating the crack width tensioning stiffening is taken into account.

8.2 Structural verification

The sections were the terminal structure foundation has been verified are presented in the sketchs shown below:

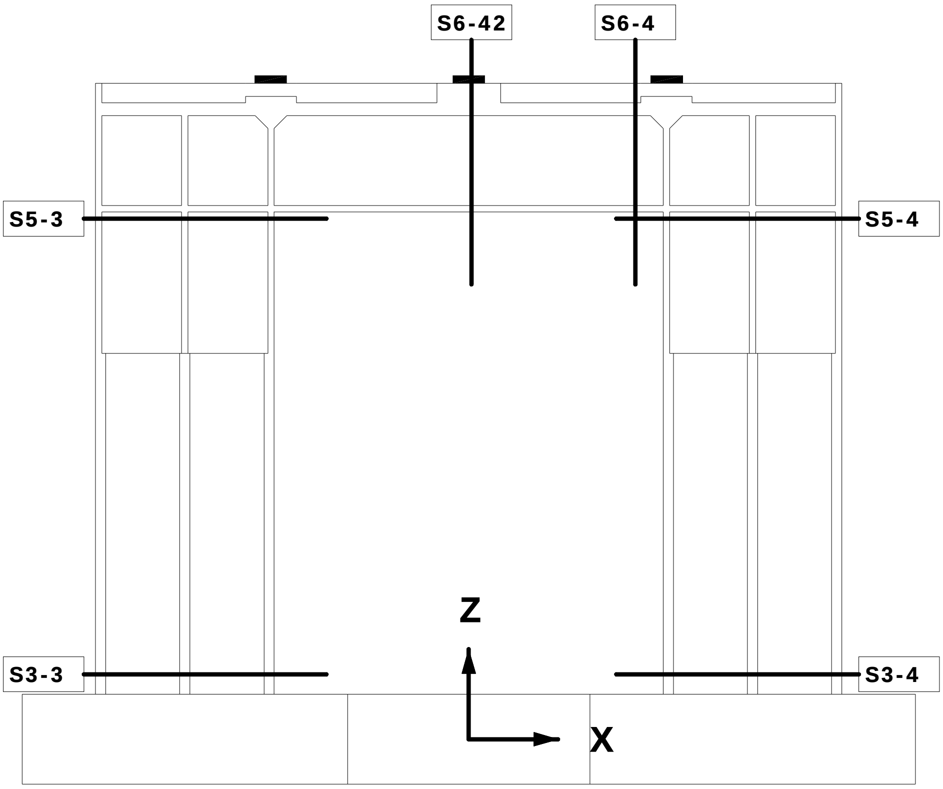


Figure 8.1 Terminal structure pier – land side

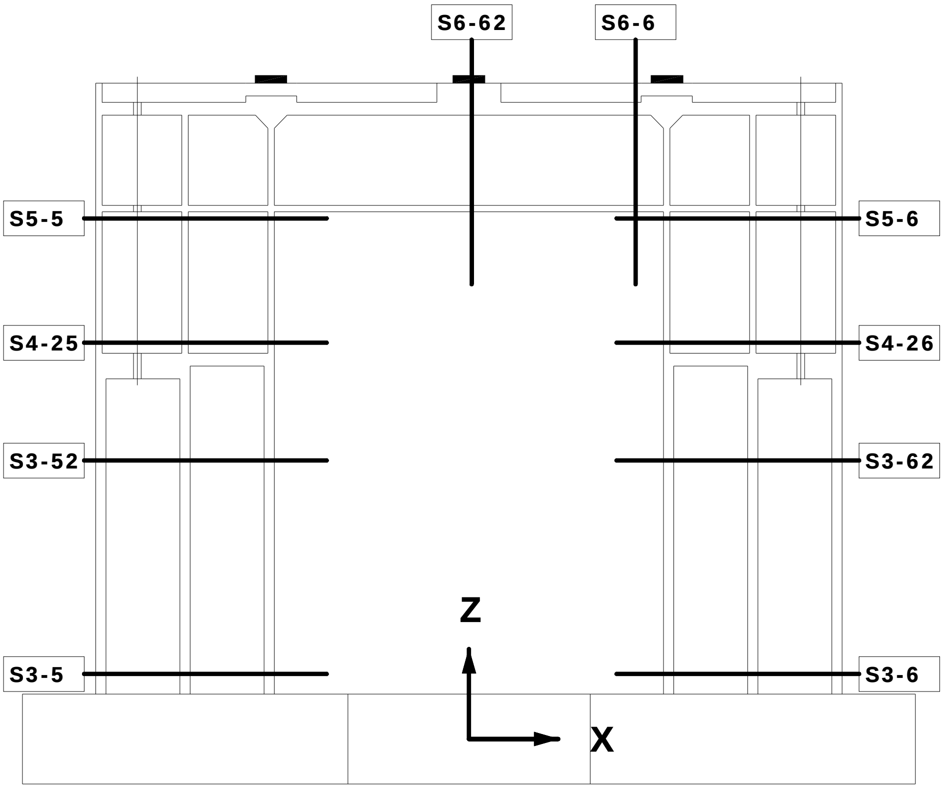


Figure 8.2 Terminal structure pier – sea side

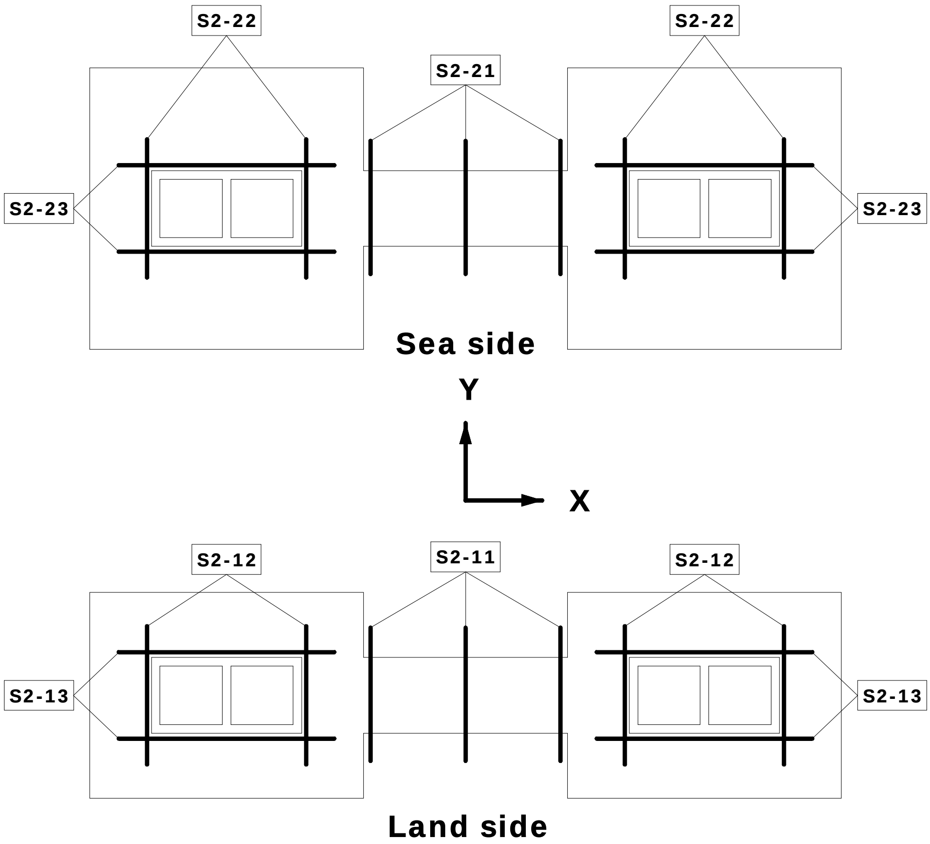


Figure 8.3 Terminal foundation

The above indicated section are the following:

- base and at the top of the piers (S3, S5).
- middle sections of piers cross beam (S3.X2).
- foundation slab (S1, S2).
- end sections of piers cross beam (S6).
- middle sections of piers cross beam (S6.X2).

These are the section necessary to design the piers for the worst load cases conditions.

8.3 Geotechnical verification

8.3.1 Introduction

Referring to the soil investigations available the terminal structure foundations Sicily side will be laying on a Coastal deposit while the foundations Calabria side will be laying on "Pezzo conglomerate".

The geotechnical verifications that have been carried out are:

- Overturning verification
- Q_{lim} verification

8.3.2 Overturning verification

A superficial model of the foundation has been considered. The acting forces (decomposed into vertical and horizontal components) and their application distances are determinate respect to a rotation point. The loads (permanent and variable) have been multiplied by the partial coefficients defined by the NTC 08.

The overturning moment M_{rib} and the M_{stab} are then calculated to determine the safety coefficient for the overturning: $F = M_{stab} / M_{rib}$

8.3.3 Q_{lim} verification

For the Q_{lim} verification the contact surface soil-foundation has been considered.

The following 3 possible collapse mechanism which have been studied:

- General collapse for spread foundations
- Local collapse for spread foundations
- Shear collapse

A quantitative indication of the occurrence of one of the previous mechanism is given by the solutions of Vesic, where an index of rigidity is given for sandy soil:

$$I_R = \left[\frac{G}{c' + \sigma_v' \tan\varphi'} \right]$$

where: G is shear modulus, q is effective pressure underneath the foundation at elevation B/2 (B = width of foundation).

This value is compared with critical values which can be found in literature and that are function of the dimensions of the foundations.

If I_R is bigger than the literature values, the conventional formulas can be used, otherwise the following coefficients need to be evaluate to take into account:

$$\Psi_q = \Psi_\gamma = \exp\left[\left(0{,}6\frac{B}{L} - 4{,}4 \right) \tan\varphi' + \frac{3{,}07 sen\varphi' \log(2I_R)}{1 + sen\varphi'} \right] = 0{,}41$$

$$\Psi_c = \Psi_q - \frac{1 - \Psi_q}{N_q tg\varphi'} = 0{,}35$$

the following formula is used for Qlim calculation:

$$q\lim = c' \cdot N_c \cdot s_c \cdot d_c \cdot b_c \cdot g_c \cdot i_c + q_0 \cdot N_q \cdot s_q \cdot d_q \cdot b_q \cdot g_q \cdot i_q + 0.5 \cdot \gamma' \cdot B' \cdot N_\gamma \cdot s_\gamma \cdot d_\gamma \cdot b_\gamma \cdot g_\gamma \cdot i_\gamma$$

where:

$N_q = e^{\pi tg\varphi} tg^2(45 + \varphi'/2)$; $N_c = (N_q - 1)\cot g\ \varphi$; $N_\gamma = 2(N_q + 1) tg\ \varphi$

$$S_q = 1 + \frac{B}{L} \tan \varphi'$$

$$S_c = 1 + \frac{B}{L} \frac{N_q}{N_c}$$

$$S_\gamma = 1 - 0,4 \frac{B}{L}$$

$$i_q = \left(1 - T / \left(N + B' \cdot c' \cdot \cot g\varphi' \right) \right)^m;$$

$$i_c = i_q - \left(1 - i_q\right) / \left(N_q - 1\right);$$

$$i_\gamma = \left(1 - T / \left(N + B' \cdot c' \cdot \cot g\varphi' \right) \right)^{m+1}$$

The effective equivalent area needs then to be evacuate, by introducing in the calculation the eccentricity along B and L due to the application of the bending moments ML e MB. The following dimensions for the foundation area need to be used in the calculation B'=B-2eB and L'=L-2eL, where e is the eccentricity.

The safety coefficient is given by the following:

$$F = \frac{q \lim}{qes}$$

where:

$$qes = N / \left(B' \cdot L \right)$$

and N is the vertical load and L is the length of the foundation.

9 Verification

This section summarizes the governing load combinations and the design verifications that have been completed for the terminal structure components. Design principles are described as necessary to clarify the effects of the governing load combinations and the verification results, however, the General Design Principles (CG.10.00-P-RG-D-P-SV-S8-00-00-00-00-01_C) should be consulted for more detailed descriptions and bases for the verification procedures.

It is noted that earthquake loading is entirely governing for the substructure for the Terminal Structures and in the serviceability state the substructure is only stressed lightly, i.e. crack widths are found to be nil. It is noted also that is the land side which reports the highest utility ratios.

Results of design verifications can be found in the following reports:

- “CG1002-P-CL-D-P-ST-S6-00-00-00-00-01_C_Design Report Sicilia Terminal Foundation”.
- “CG1002-P-CL-D-P-ST-S6-00-00-00-00-02_C_Design Report Calabria Terminal Foundation”.

9.1 Terminal Structure, Sicilia side

9.1.1 Terminal Substructure

The terminal substructure is found to be entirely governed by the seismic load and the following is summarised from the verifications.

Position	Section	Position	Ratio (w/o EQ)	Ratio (incl. EQ)
Land side	S3-3 + S3-4	Base	28.104	1.299
Land side	S5-3 + S3-4	Top	4.461	2.498
Sea side	S3-5 + S3-6	Base	21.571	1.126
Sea side	S3-52 + S3-62	Mid	13.488	1.590
Sea side	S4-25 + S4-26	Mid	23.497	1.556
Sea side	S5-5 + S5-6	Top	9.801	2.839

Table 9-1: Piers - Flexural bending - Capacity demand ratio

Position	Section	Position	Ratio (w/o EQ)	Ratio (incl. EQ)
Land side	S6-4 + S6-42	End	2.904	1.198
Sea side	S6-6 + S6-62	End	3.929	1.246

Table 9-2: Cross beam - Flexural bending - Capacity demand ratio

The capacity-demand ratio in relation to flexural bending for non-EQ is overall considerably above unity and is satisfactorily verified in the GEOSTRU output. Likewise the shear is reported as satisfactorily verified.

In accordance with the low utility ratios in flexural bending the SLS crack widths are calculated to be nil and thus is also verified satisfactorily.

EQ is entirely governing for the design and capacity-demand ratios are found varying in the range between 1.126 and 2.839 for the piers and between 1.198 and 1.246 for the cross beam.

It is noted that it is the land side which reports the highest utility ratios.

For tie-down diaphragm is reported the following.

Position	Position	Ratio (incl. EQ)	Crack widh
Sea side	Inside pier	1.746	Wmax = 0.1 mm

Table 9-3: Tie-down diaphragm - Flexural bending - Capacity demand ratio

9.1.2 Terminal foundation

The terminal foundation is found to be entirely governed by the seismic load and the following is summarised from the verifications.

Position	Section	Position	Ratio (w/o EQ)	Ratio (incl. EQ)
Land side	S2-11	Link	1.050	2.579
Land side	S2-12	Transverse	4.436	6.704
Land side	S2-13	Longitudinal	8.462	1.558
Sea side	S2-21	Link	1.728	1.760

Sea side	S2-22	Transverse	7.121	5.974
Sea side	S2-23	Longitudinal	8.542	1.039

Table 9-4: Foundation slabs - Flexural bending - Capacity demand ratio

The capacity-demand ratio in relation to flexural bending for non-EQ is overall considerably above unity and is satisfactorily verified in the GEOSTRU output. Likewise the shear is reported as satisfactorily verified.

In accordance with the low utility ratios in flexural bending the SLS crack widths are calculated to be nil and thus is also verified satisfactorily.

Both EQ and static ULS are governing for the design and capacity-demand ratios are found varying in the range between 1.039 and 8.542.

9.2 Terminal Structure, Calabria side

9.2.1 Terminal substructure

The terminal substructure is found to be entirely governed by the seismic load and the following is summarised from the verifications.

Position	Section	Position	Ratio (w/o EQ)	Ratio (incl. EQ)
Land side	S3-3 + S3-4	Base	8,17	4,09
Land side	S5-3 + S3-4	Top	3,83	3,34
Sea side	S3-5 + S3-6	Base	11,34	1,51
Sea side	S4-25 + S4-26	Mid	6,45	1,35
Sea side	S5-5 + S5-6	Top	9,04	3,39

Table 9-5: Piers - Flexural bending - Capacity demand ratio

Position	Section	Position	Ratio (w/o EQ)	Ratio (incl. EQ)
Land side	S6-4 - S6-42	End - Mid	2,89	1,32
Sea side	S6-6 - S6-62	End - Mid	4,67	1,76

Table 9-6: Cross beam - Flexural bending - Capacity demand ratio

The capacity-demand ratio in relation to flexural bending for non-EQ is overall considerably above unity and is satisfactorily verified in the GEOSTRU output. Likewise the shear is reported as satisfactorily verified.

In accordance with the low utility ratios in flexural bending the SLS crack widths are calculated to be nil and thus is also verified satisfactorily.

EQ is entirely governing for the design and capacity-demand ratios are found in a range varying between 4,09 and 1,32.

For tie-down diaphragm is reported the following.

Position	**Position**	**Ratio (incl. EQ)**	**Crack widh**
Sea side	Inside pier	1.746	W max = 0.1 mm

Table 9-7: Tie-down diaphragm - Flexural bending - Capacity demand ratio

9.2.2 Terminal foundation

The terminal foundation is found to be entirely governed by the seismic load and the following is summarised from the verifications.

Position	**Section**	**Position**	**Ratio (w/o EQ)**	**Ratio (incl. EQ)**
Land side	S2-11 – S2-14	Link	1,05	8,85
Land side	S2-12	Transverse	1,92	9,39
Land side	S2-13	Longitudinal	8,27	3,86
Sea side	S2-21 – S2-24	Link	2,40	4,92
Sea side	S2-22	Transverse	3,62	12,52
Sea side	S2-23	Longitudinal	10,16	1,10

Table 9-8: Foundation slabs - Flexural bending - Capacity demand ratio

The capacity-demand ratio in relation to flexural bending for non-EQ is overall above unity and is satisfactorily verified in the GEOSTRU output. Likewise the shear is reported as satisfactorily verified.

In accordance with the low utility ratios in flexural bending the SLS crack widths are calculated to be nil and thus is also verified satisfactorily.

Both EQ and static ULS are governing for the design and capacity-demand ratios are found in the range varying between 12,52 and 1,05.

10 Summary

Earthquake loading is entirely governing for the substructure for the Terminal Structures and in the serviceability state the substructure is only stressed lightly, i.e. crack widths are found to be nil.

Terminal Structure, Sicilia side

For the ULS limit state with EQ the highest utility ratio in flexural bending is 1.198 in the cross beam of the pier, and 1.126 in the pier sea side and is this verified satisfactorily as is the shear capacity.

Foundation is verified and the for the ULS limit state with EQ the highest utility ratio in flexural bending is 1.039 in the slab, land side.

Terminal Structure, Calabria side

For the ULS limit state with EQ the highest utility ratio in flexural bending is 1.32 in the cross beam of the pier, and 1.51 in the pier land side and is this verified satisfactorily as is the shear capacity.

Foundation is verified and the highest utility ratio in flexural bending is 1.05 the ULS limit state in static condition in the slab sea side.

It is noted that it is the land side which reports the highest utility ratios.

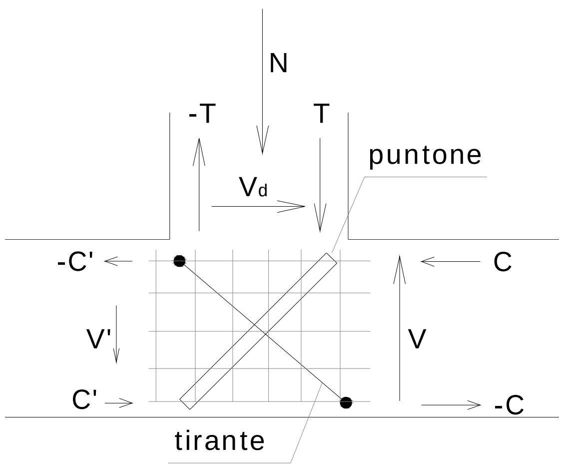
For tie-down diaphragm is reported that Earthquake loading governing, and sea side diaphragm is more stressed. The highest utility ratio in flexural bending is 0.57.

The serviceability state is only stressed lightly, i.e. maximum crack widths in the tie-down diaphragm are found to be 0.1 mm.

Appendix 1 – Joint Shear Calculation

The joint shear mechanism, occurring in the node plinth-pier base, is characterized by the convergence in the node of the forces as shown in the figure below.

In the figure N, T, T' and Vd are respectively the Normal force, the push and pull forces due to the bending moment, and the shear force acting at the base of the pier, while V, C and –C are respectively the shear forces and the compression and traction resultant forces acting in the concrete and in the reinforcement bars in correspondence of the base of the pier, due to the effect of terrain reactions and finally V' C' e -C' are the equivalent forces acting in the other side of the node due to the effect of the self-weight of the plinth cantilever part respect to the section at the base of the pier.



The force acting in the truss in traction is absorbed by the horizontal and vertical reinforcement bars in the node.

The above described forces are projected along the directions of the trusses in compression and in tension, obtaining a null resultant for both the vertical components of the forces and for both compression and tension horizontal forces due to bending moments, occurring in the plinth section in correspondence of the pier base section.

The shear V_d at the base of the pier is projected along the direction of the truss in traction to verify the reinforcement in the node.

In the case considered, the worst condition occurs for the load combination which maximizes the moments (and so also the shear) in the longitudinal direction of the deck; for this load condition the assessment is reported in this document. In particular, as the plinth is 7.0m high and the pier is of 7.0 m wide, the truss in traction and of the truss in compression form an angle of 45° with the horizontal.

Considering the forces acting at the base of the piers, the worst condition occurs in correspondence of the pier of the terminal structure Sicily side, sea side, where the value of the base pier shear is V_d= 67465kN.

In the verification, it is considered a section given by the section diffused at 45° of the longitudinal width of the pier is 7.0 m on the middle section of the foundation, where the reinforcement is made up of vertical bars ϕ32/90x90 and horizontal bars ϕ32/30x90, which guarantee a resistant shear value of:

Vertical Reinforcement

$V_{Rsd} = (0.9 \cdot d \cdot A_{sw} \cdot f_{yd}/s)=$ 32808.1 KN

Horizontal Reinforcement

$V_{Rsd} = (0.9 \cdot d \cdot A_{sw} \cdot f_{yd}/s)=$ 100611.4 KN

These values are projected along the directions of the trusses furnishing a total resistant value of:

V_{Rd}= 32808.1/√2+100611.4/√2= 94341.8 kN.

The verification is satisfied:

$V_{Rd} > V_d$