Stretto di Messina

Concessionaria per la progettazione, realizzazione e gestione del collegamento stabile tra la Sicilia e il Continente
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EurolinK

PONTE SULLO STRETTO DI MESSINA

PROGETTO DEFINITIVO

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Titolo del documento	General Design Principles for Terminal Superstructures	

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1 Introduction

This report provides descriptions of the terminal superstructures design considerations and of the the principles of the structural analyses, including the assumptions that have been made. The design criteria, including philosophy and code references, by which the following components of the terminal structure superstructure have been designed:

- Transverse steel truss
- Longitudinal webs
- Transverse webs
- Concrete deck slab

The design is based on that shown in the Tender Design with this phase introducing the following changes to the Tender Design:

- Overall width increased from to 59708 mm to 69870 mm with the increase equally distributed on the two service lanes to accommodate the Suspended Deck dilation inside the facade of the Terminal Structure
- Outwards sloping 2 % cross fall of the deck
- Articulation system has been modified by removing the hydraulic buffers.

1.1 Scope

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

1.2 Report Outline

This report is organized into the following sections:

- *Section 1* includes an introduction, provides a list of reference materials, including design specifications, design codes, material specifications, reference drawings and complementary reports;

- *Section 2* provides definitions for terms that are commonly used in referencing particular terminal structure components;
- *Section 3* describes the three limit states that are considered in the terminal structure superstructure design, serviceability, ultimate and structural integrity;
- *Section 4* provides descriptions of the materials that are used for each terminal structure superstructure component;
- *Section 5* provides descriptions of the terminal structures superstructures design consideration;
- *Section 6* describes the principles of the structural analyses, including the structural model descriptions and the assumptions that have been made;
- *Section 7* describes the design criteria, including philosophy and code references, by which the terminal structure superstructure components are verified for the limit states;
- *Section 8* describes the articulation system.

1.3 Basic studies

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

1.3.1 Design Basis

New technical standards for construction, NTC08, and new Italian railway codes, RFI DTC-ICI-PO SP INF are now standard in Italy. These codes are based on the Eurocodes and 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 the 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.

1.4 References

1.4.1 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.

CG.10.00-P-RG-D-P-GE-00-00-00-00-00-02, “Application Manual to Design Basis, Structural, Annex”

1.4.2 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"

1.4.3 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.

1.4.4 Drawings

Terminal superstructure: Sicily side	
CG1002-P-AX-D-P-SV-S8-VS-00-00-00-01	Terminal Structures, Sicilia / General Arrangement
CG1002-P-PX-DP-SV-S8-VS-00-00-00-01	Terminal Structures, Sicilia / Deck, longitudinal plan and front view
CG1002-P-BX-DP-SV-S8-VS-00-00-00-01	Terminal Structures, Sicilia / Steel structures 1
CG1002-P-BX-DP-SV-S8-VS-00-00-00-02	Terminal Structures, Sicilia / Steel structures 2
CG1002-P-BX-DP-SV-S8-VS-00-00-00-03	Terminal Structures, Sicilia / Steel structures 3
CG1002-P-BX-DP-SV-S8-VS-00-00-00-04	Terminal Structures, Sicilia / Steel structures 4
CG1002-P-BX-DP-SV-S8-VS-00-00-00-05	Terminal Structures, Sicilia / Steel structures 5
CG1002-P-BX-DP-SV-S8-VS-00-00-00-06	Terminal Structures, Sicilia / Steel structures 6
CG1002-P-BX-DP-SV-S8-VS-00-00-00-07	Terminal Structures, Sicilia / Steelwork, details 1
CG1002-P-BX-DP-SV-S8-VS-00-00-00-08	Terminal Structures, Sicilia / Steelwork, details 2
CG1002-P-BX-DP-SV-S8-VS-00-00-00-09	Terminal Structures, Sicilia / Platform, details
CG1002-P-BX-DP-SV-S8-VS-00-00-00-10	Terminal Structures, Sicilia / Deck, concrete slab
CG1002-P-DX-DP-SV-S8-VS-00-00-00-01	Terminal Structures, Sicilia / Bearing and expansion joints arrangement, plan
Terminal superstructure: Calabria side	
CG1002-P-AXDPSV-S8-VC-00-00-00-01	Terminal Structures, Calabria / General Arrangement
CG1002-P-AXDPSV-S8-VC-00-00-00-02	Terminal Structures, Calabria / Deck, longitudinal plan and front view
CG1002-P-AX-DP-SV-S8-VC-00-00-00-03	Terminal Structures, Calabria / Steel structures 1
CG1002-P-AX-DP-SV-S8-VC-00-00-00-04	Terminal Structures, Calabria / Steel structures 2
CG1002-P-AX-DP-SV-S8-VC-00-00-00-05	Terminal Structures, Calabria / Steel structures 3
CG1002-P-AX-DP-SV-S8-VC-00-00-00-06	Terminal Structures, Calabria / Steel structures 4

CG1002-P-AX-DP-SV-S8-VC-00-00-00-07	Terminal Structures, Calabria / Steel structures 5
CG1002-P-AX-DP-SV-S8-VC-00-00-00-08	Terminal Structures, Calabria / Steel structures 6
CG1002-P-AX-DP-SV-S8-VC-00-00-00-09	Terminal Structures, Calabria / Steelwork, details 1
CG1002-P-AX-DP-SV-S8-VC-00-00-00-10	Terminal Structures, Calabria / Steelwork, details 2
CG1002-P-AX-DP-SV-S8-VC-00-00-00-11	Terminal Structures, Calabria / Platform, details
CG1002-P-AX-DP-SV-S8-VC-00-00-00-12	Terminal Structures, Calabria / Deck, concrete slab
CG1002-P-AX-DP-SV-S8-VC-00-00-00-13	Terminal Structures, Calabria / Bearing and expansion joints arrangement, plan

1.4.5 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

Design report - Bridge Bearings Doc. No.: CG1000-P-CL-D-P-SS-A0-AP-00-00-00-01

Design report - Expansion joints Doc. No.: CG1000-P-CL-D-P-SS-A0-AM-00-00-00-01

Performance Specification - Bridge Bearings Doc. No.: CG1000-P-SP-D-P-SS-A0-AP-00-00-00-01

Performance Specification - Buffers Doc. No.: CG1000-P-SP-D-P-SS-A0-AM-00-00-00-01

Performance Specification - Expansion joints, Railway Doc. No.: CG1000-P-SP-D-P-SS-A0-AM-00-00-00-02

Performance Specification - Expansion joints, Roadway Doc. No.: CG1000-P-SP-D-P-SS-A0-AM-00-00-00-03

2 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.

Transverse Webs – 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.

3 Limit States

This section describes the limit states and corresponding performance requirements governing the proportioning of the terminal structures components, in accordance with the project design basis PG0025 and NTC08. The performance of terminal structure components is verified at Serviceability Limit States (1 and 2) Ultimate Limit States and Fatigue Limit States.

3.1 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 PG0025 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 3-1.

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 3-1: SLS performance requirements.

Fatigue related SLS are addressed in Section 3.3.

3.2 Ultimate Limit States

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

- 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.
- 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 PG0025 Section 3.1 specifies the performance requirements for the structure under ultimate or rare loads. The performance requirements are listed in Table 3-2.

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 3-2: ULS performance requirements.

Fatigue related ULS are addressed in Section 3.3.

3.3 Fatigue Limit States

NTC08 Sections 2.2.1 and 2.2.2 do not distinguish fatigue limit states (FLS) from serviceability and ultimate limits states with similar consequences and performance requirements. However, in NTC08 Section 4.2.2.1 and in this report, FLS are distinguished because the loads and load combinations used for verification are different, as are the means by which the elements are verified. NTC08 Sections 2.2.1 and 2.2.2 define the following fatigue related SLS and ULS that are to be evaluated in a structural design:

- Damage caused by fatigue that could compromise durability (SLS).
- Failure of frames and joints due to fatigue (ULS).

4 Materials

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

4.1 Concrete and reinforcement

4.1.1 Concrete

All structural concrete for the deck slab to be Grade C35/45 in accordance with EN 206-1:2001.

Concrete Type	**Deck Slab**
Concrete Grade	**C35/45**
Time to develop Strength	28 days
Environmental class	XC4+XS1
Consistency Class	S4/S5
Max aggregate size	25 mm for the deck and 10mm for predalles
Cement	CEM III/B in accordance to EN 197-1
Max Total Alkali content of cement	0.6%
Min. Cement content in Kg/m^3	360
Max W/C ratio	0.42
Chloride content class	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	25
Nominal cover to stainless steel	-

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 4-1: Concrete mechanical properties

4.1.2 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.

4.2 Structural Steel

Terminal structures are fabricated from Grade S 355 ML structural steels, produced in accordance with EN 10025-4. The steel is assumed to have the mechanical properties listed in Table 4-2, in accordance with NTC08 Section 11.3.4.1.

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

Table 4-2: Structural steel mechanical properties

For thicknesses higher than 50 mm, the reference stresses have to be reduced to 335/450 MPa, according to NTC08.

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

- Elastic modulus: 210000 MPa
- Poisson's ratio: 0.3
- Shear modulus: 80769 MPa
- Coefficient of thermal expansion: 12 x 10 $^{-6}$ / °C $\alpha = 12 \times 10^{-6}/°C$
- Density: 7850 kg/m ^{3} $\rho = 7850\ kg/m^3$

The material partial factors (safety coefficients) used to verify structural steel elements are in accordance with NTC08 Section 4.2.4.1.1, 4.2.4.1.4 and are listed in Table 4-3.

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

Table 4-3: Material partial factors for structural steel.

4.3 High Strength Bolts

High strength structural bolts of Grade 8.8 bolts are used for connections of all non-structural components to the terminal structures. High strength bolts are assumed to have the mechanical properties listed in Table 4-4, in accordance with NTC08 Section 11.3.4.6.1.

Grade	Yield Strength, f_{yb} (MPa)	Tensile Strength, f_{tb} (MPa)
8.8	649	800

Table 4-4: Structural bolt mechanical properties.

4.4 Shear Studs

Shear studs are manufactured from low carbon steel with a minimum yield point of 350 N/mm^2 and ultimate tensile strength of 450 N/mm^2 and an elongation of 15 %. The studs should be specified with a shank diameter (d) of 19 mm.

4.5 Welding Consumables

The material partial factors (safety coefficients) used to verify welded connections are in accordance with NTC 2008 Section 4.2.8.1.1. and are listed in Table 4-5.

Verification	***Partial Factor***
Resistance to weldings to partial penetration and fillet weld	$\gamma_{M2} = 1.25$
ULS slip resistance	$\gamma_{M3} = 1.25$
SLS slip resistance	$\gamma_{M3} = 1.10$

Table 4-6: Material partial factors for welded connections.

4.6 Stainless Steel

Stainless steel is of grade AISI 316L.

5 Superstructure Design Considerations

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 the Terminal Structure is linked to the suspension bridge and to a viaduct linking the main land.

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 and the viaduct Calabria land side also land 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, for the Sicilia side and are 60.870 m and 71.200 m, for the width and length respectively, for the 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 a platform is located on either side of the railway girder for evacuation if necessary. The platform is continuous over the entire Terminal Structure length.

The crash barriers are continuous over the entire bridge length and provide a safe barrier for the road traffic. The wind screen on the service lane reduces the wind speed across the bridge.

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.

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.

The main components are described in the following where figures illustrate the layout of the Terminal Structure.

Below is shown the general layout in Figure 5-1.

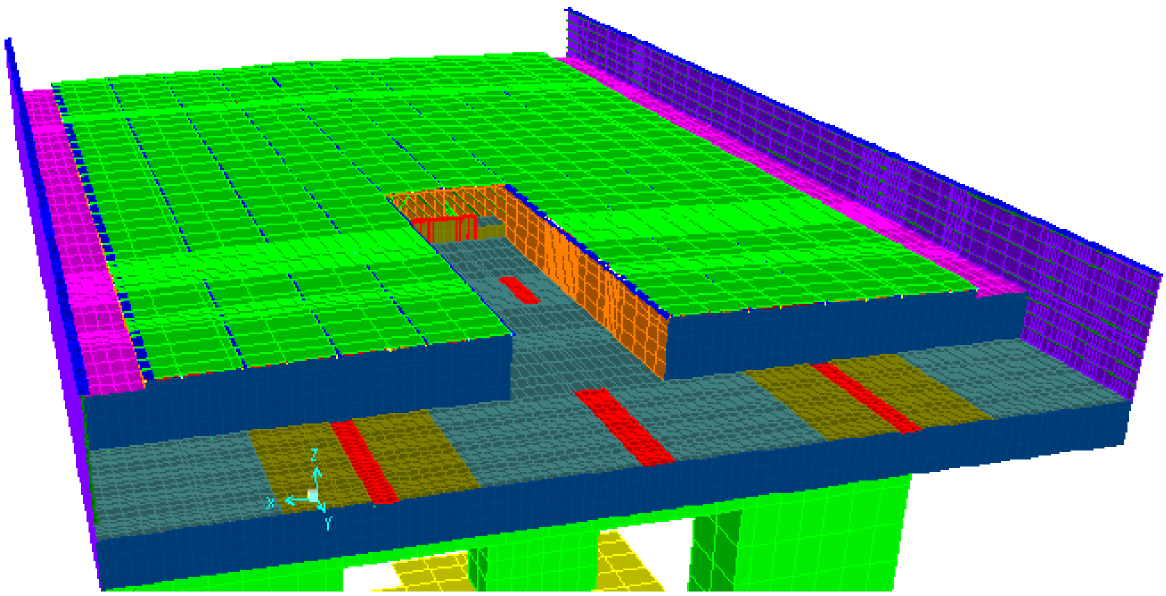


Figure 5-1: Isometric view - Superstructure general layout

5.1 Steel Box

The steel box is composed of an open steel frame with is closed by use of steel plates (typically t = 10 mm) which are provided with appropriate stiffeners to ensure the stability of the plates. The closing steel plates increase in thickness at locations where concentrated loads are present, i.e. typically at the bearings.

The frame is composed of I-shaped profiles (typically IPE600, HE500B and HE600A) which are also applied as half-sections. The half-sections comprise a vertical grid.

At the bottom half-sections are connecting the vertical grid in the transverse direction and the closing steel plates have stiffeners in the longitudinal direction.

At the top the vertical grid is connected by both HEB500-profiles and steel plates (300×40 THK) in the transverse direction and steel plates (500×40 THK) in the longitudinal direction. These top steel elements carry the concrete deck and have shear stud connectors attached.

Inside the steel box are also a transverse truss and longitudinal and transverse webs which are described further in detail in the following sections.

The layout is shown in Figure 5-2:

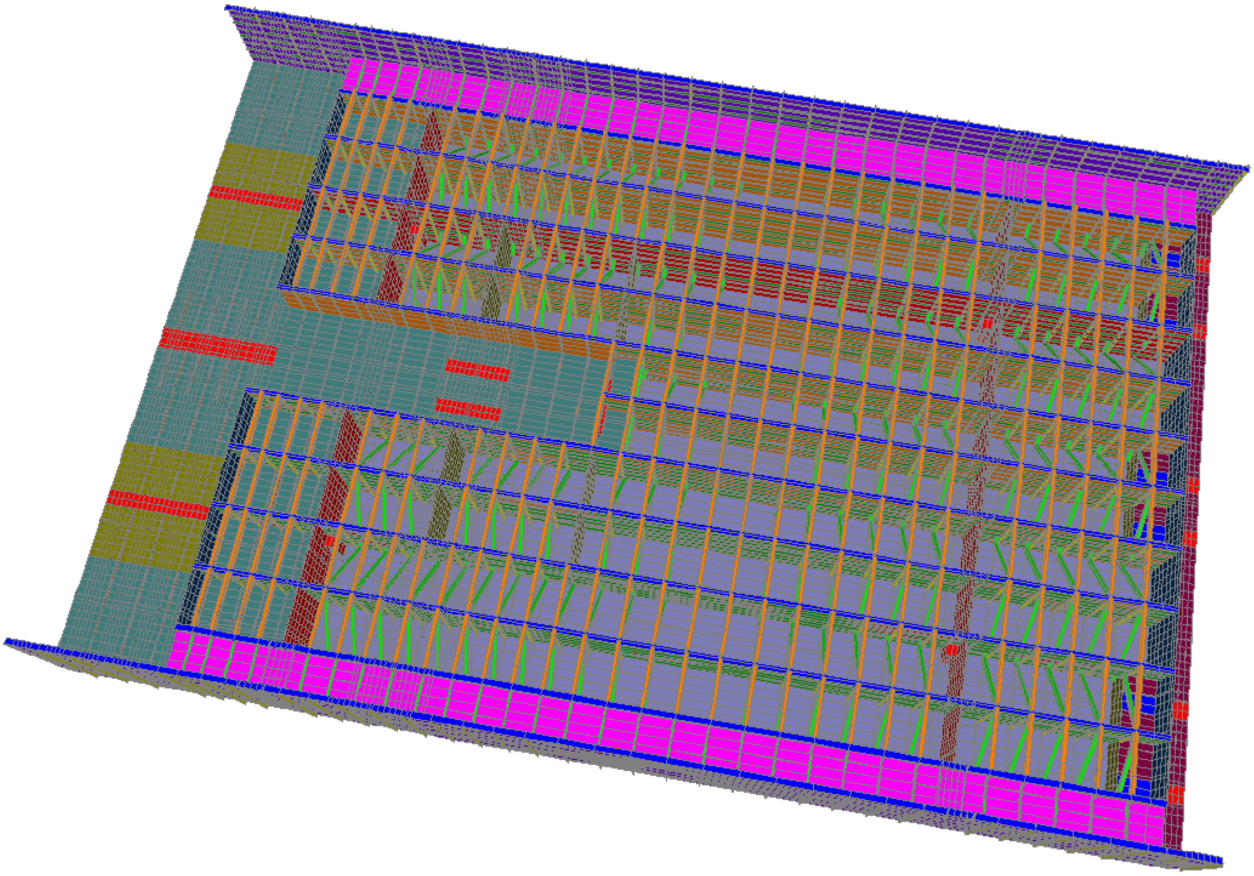


Figure 5-2: Isometric view - Layout of steel box

Initially the steel box carries its own weight including the in-situ cast concrete and composite action is applicable after the concrete has achieved its final strength.

5.2 Transverse steel truss

In order to stiffen the steel frame a truss structure in the transverse direction is applied. The truss is composed of HE300B-profiles and is shown in Figure 5-3.

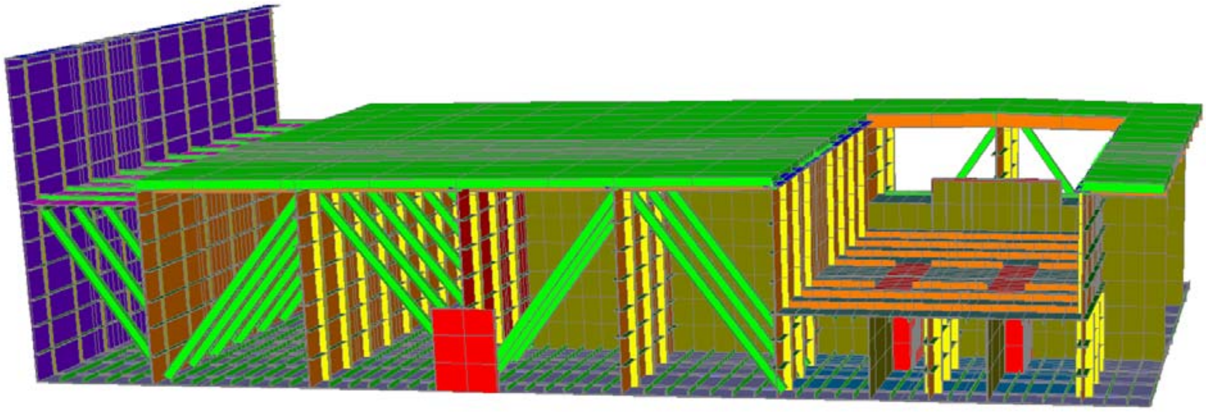


Figure 5-3: Isometric view - Transverse steel truss

5.3 Longitudinal webs

10 longitudinal webs run in the entire length inside the Terminal Structure. These are typically of thickness t = 15 mm. However, those longitudinal webs above the bearings which support the superstructure at have an increased thickness of t = 35 m.

In correspondence of the bearings of the superstructure on the piers land side both transversal and longitudinal webs are thickened. The longitudinal webs with t= 35 mm. are thickened till t= 50 mm. for a length of ~ 5 m. on the other hand the two webs on the right and on the left side of the bearing with t= 15 mm. are thickened till t= 20 mm. for a length of ~ 5 m.

The longitudinal webs divide the Terminal Structure into longitudinal compartments and openings are provided in order access transversely through the longitudinal webs.

The longitudinal webs can be seen in the figures above.

5.4 Diaphragms

Heavy diaphragms (t = 40 mm) run across the Terminal Structure above the bearings which support the superstructure. The transverse webs in correspondence of bearings between piers and superstructures land side is 50 mm thick.

The diaphragms divide the Terminal Structure into transverse compartments and openings are provided in order access longitudinally through the transverse webs.

Supplementary diaphragms (t = 15 mm and 25 mm) are applied on either side of the trench where the suspended deck rail girder extends into the Terminal Structure. However, these diaphragms are limited in the transverse direction.

The diaphragms are stiffened to ensure the plate stability; they can be seen in the figures above.

5.5 Concrete Deck Slab

The mildly reinforced concrete deck slab has a thickness varying between 300 mm and 400 mm including 50 mm of pre-cast elements and with the thicker part towards the rail tracks. This part of the deck has no need for asphalt surfacing and is flush with the roadway surface whereby a constant transverse slope of 2 % is maintained in order to drain the water across the deck without obtrusions. The road surfacing consists of 110 mm asphalt.

The concrete deck cross section is shown in Figure 5-4.

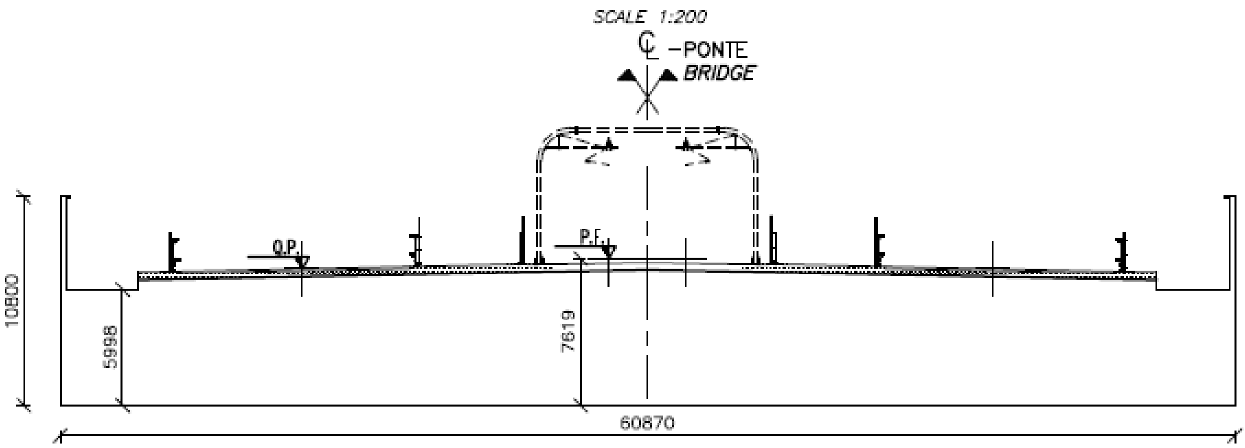


Figure 5-4: Concrete deck cross section

5.6 Shear Studs

Shear studs connect the steel frame with the concrete deck slab. Transversely shear studs are applied in a single row and distributed at 250 mm whilst in the longitudinal direction they are applied in double rows 250 mm apart and sets of two studs is distributed at 300 mm.

6 Structural Analysis

In the following a description of the modelling of the Terminal Structure is given. The terminal structures are modelled and analysed in the analysis program SAP (Structural Analysis Program). This section describes the approach to particular aspects of the structural analysis that affect the terminal structure deck design.

In the SAP semi-local FE-model, the superstructure is modelled with shell elements. This model is used for the overall verification of the structure elements. Further a local shell FE-model will be developed in order to verify local effects. Hereby it will be possible to investigate the stress flow within the deck structure in more details.

For more details about the modelling reference is made to:

- "CG.10.02-P-RX-D-P-SV-S8-VT-000-01 Specialist Technical Design Report, Sicily"
- "CG.10.02-P-RX-D-P-SV-S8-VC-000-02 Specialist Technical Design Report, Calabria."

6.1 Construction stages

Constructions phases have been taken into account for the design of the Terminal supertructure. In particular construction loads have been considered in accordance with the following stages:

- Stage 1: Concrete is poured and supported by predalles on the steel construction
- Stage 2: Composite action for live load
- Stage 3: Long term analysis to calculate the stresses due to concrete creep

Separate construction analysis phases in SAP2000 have been implemented.

6.2 Loads

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").

6.2.1 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.

6.2.2 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 supserstructure, 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.

6.2.2.1 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:

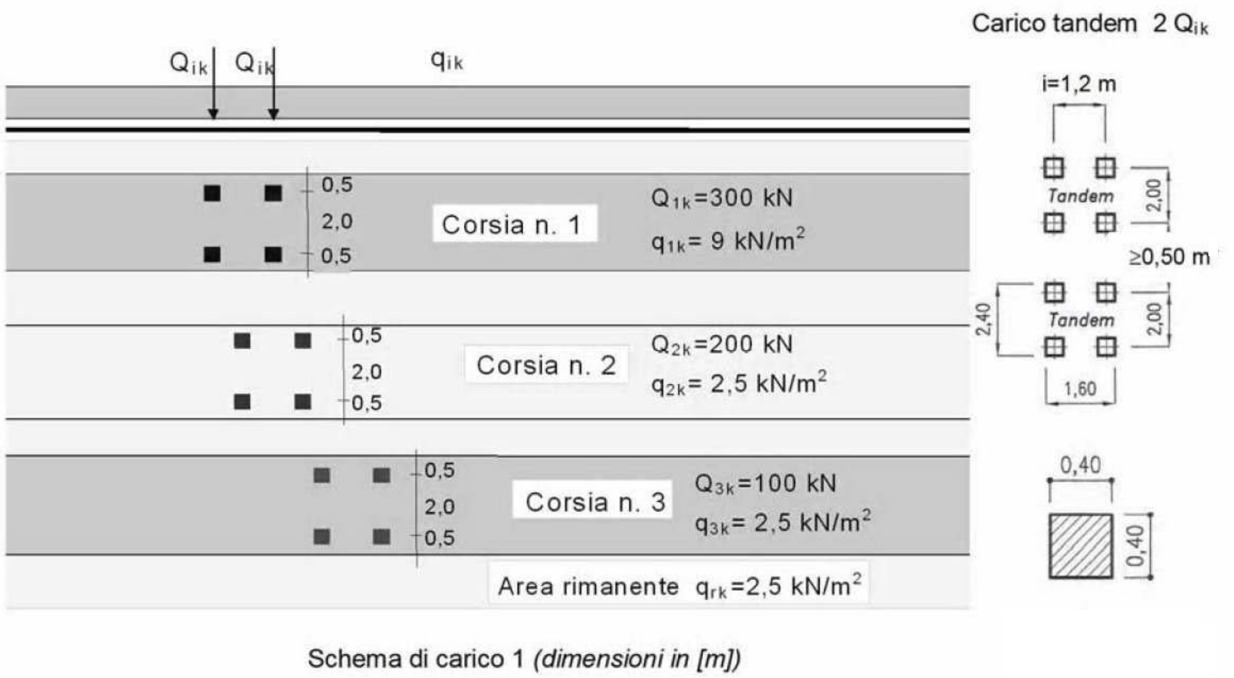


Figure 6-1: Application of Load model 1

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

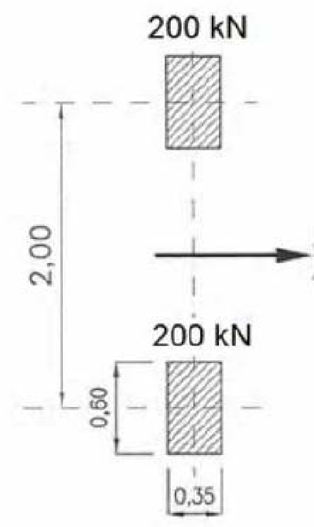


Figure 6-2: Application of model 2 (schema di carico 2)

Once the alignments of shells representing the lanes have been identified, the tandem system loads have been applied diffusing them uniformly 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.

6.2.2.2 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.

Q_{vk} = 275 kN   275kN 275kN 275 kN

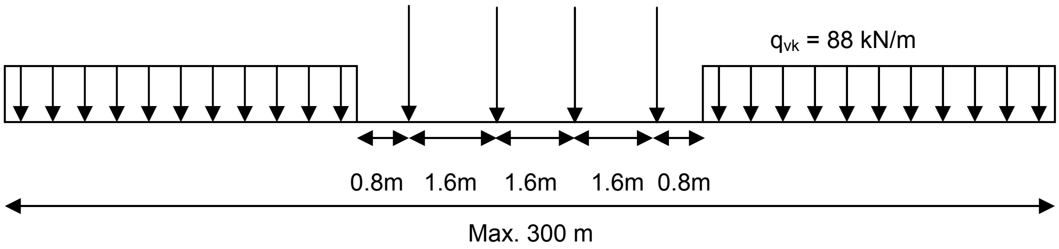


Figure 6-3: Load model 71 with α=1.10

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.

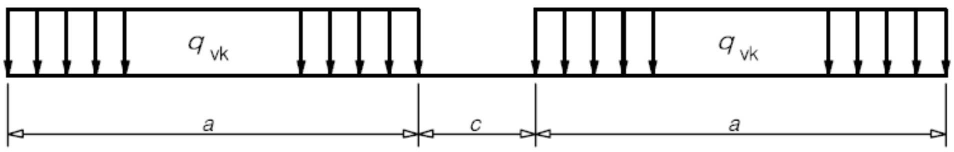


Figure 6-4: Load model SW/0 and SW/2

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:

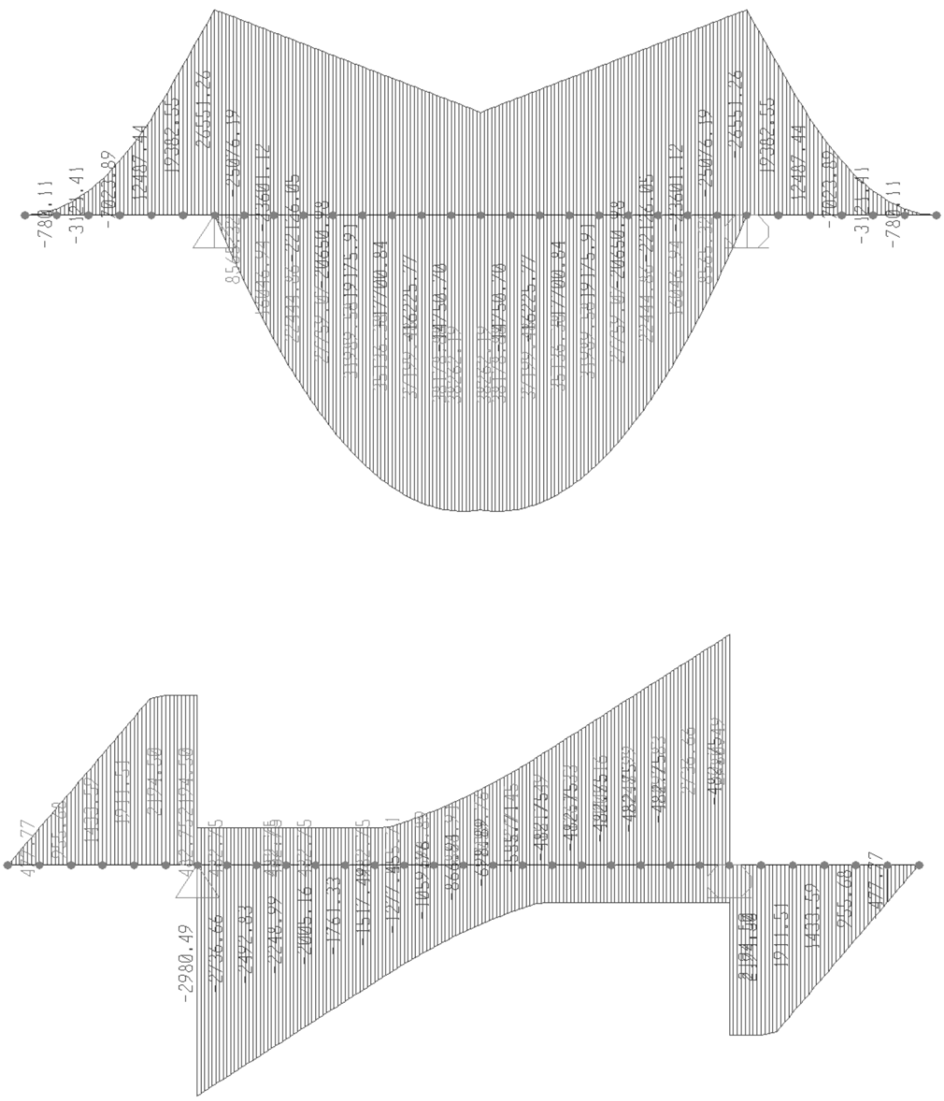


Figure 6-5: Forces induced by Load model SW/0

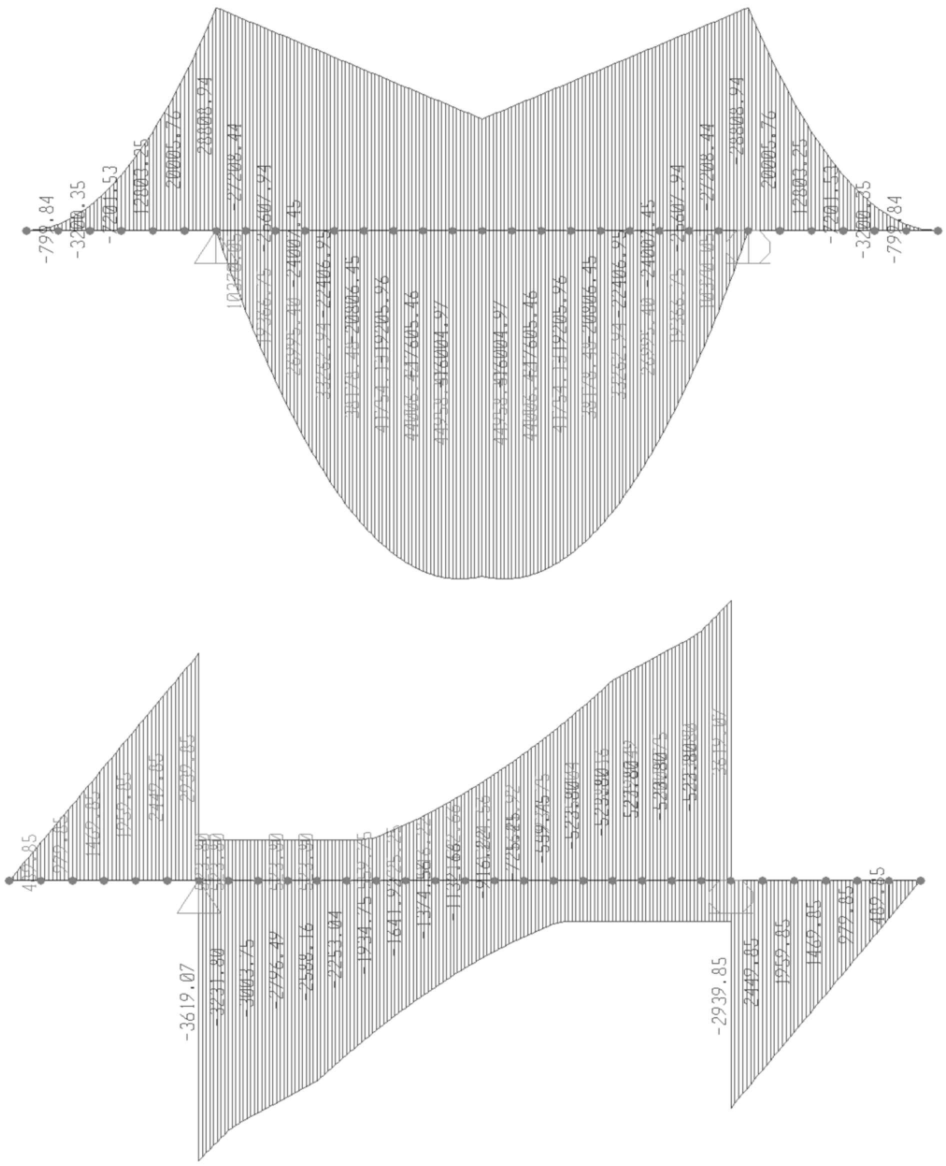


Figure 6-6: Forces induced by Load model SW/2

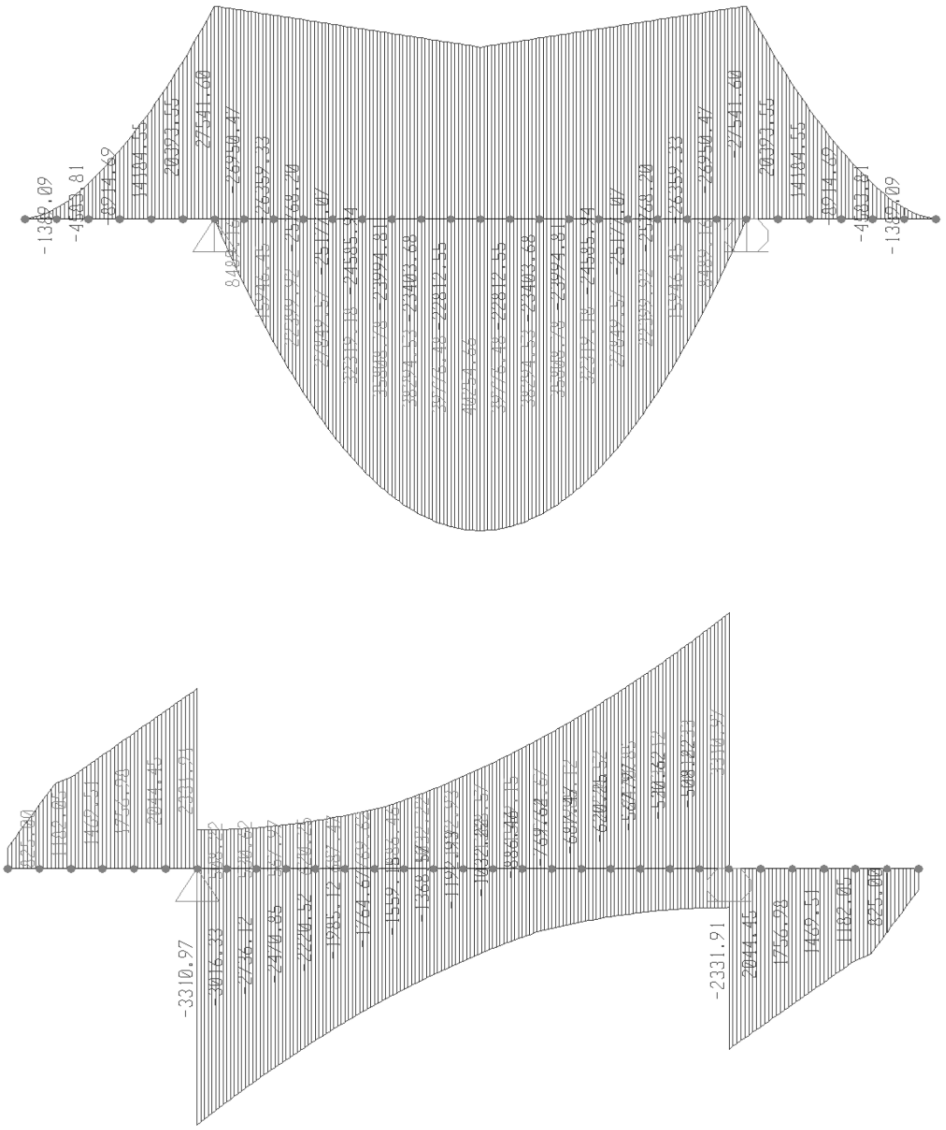


Figure 6-7: Forces induced by Load model LM71

Resulting forces from the train load models have been amplified with dynamic coefficient $\Phi = 1.06$, calculated with dynamic analyses performer in the global IBDAS model.

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 cannot 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.

6.2.3 Wind Loading

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

6.2.3.1 Static wind

The static mean wind is implemented with a vertical profile as described in the Design Criteria PG0025. 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 figure.

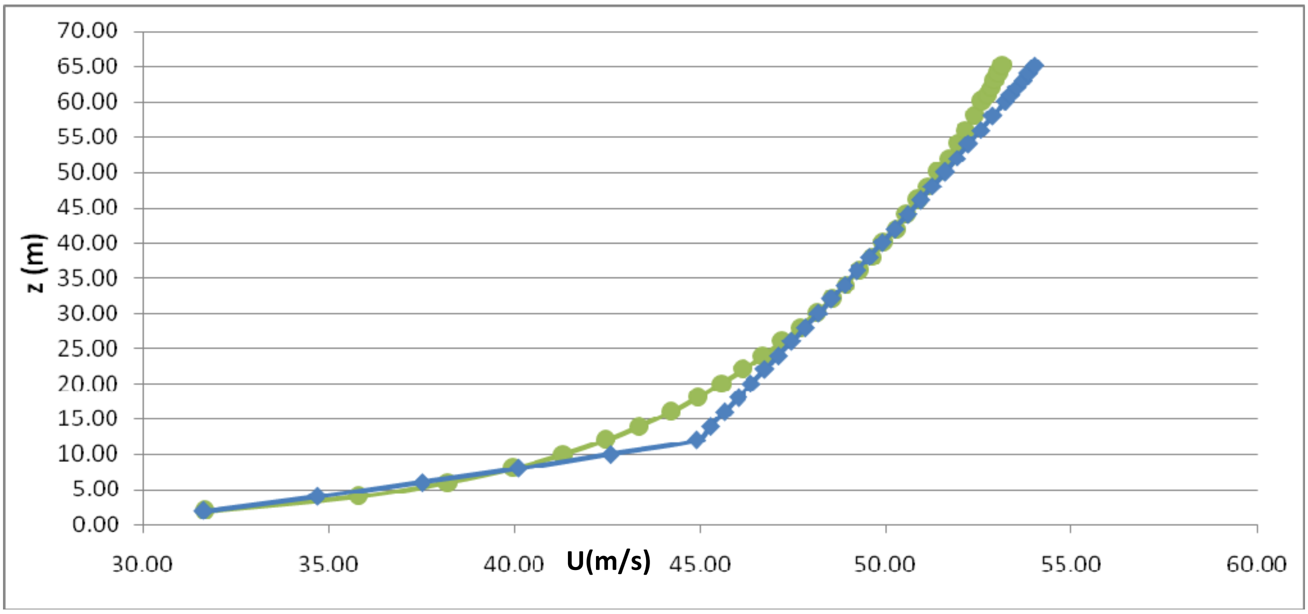


Figure 6-8: Static wind vs elevation

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.

6.2.4 Temperature Loads

Temperature Loads are implemented according to the Design Criteria PG0025, section 5.3.

Temperature variation are introduced in the model adding a joint patterns able to reproduce along the shells that variation.

6.2.5 Seismic Loads

Seismic load is implemented according to the Design Criteria PG0025, 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.

6.3 Load Combinations

The deck components are verified for the load combinations listed in the design basis (PG0025) 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 22 and Table 23, Table 24 and Table 25, respectively for SLS, ULS and SILS. 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 tables containing the used load combinations are reported in Appendix 3 of report PF0158.

6.4 Global Behaviour

The design of most terminal structure components is based on the results of the semi-local model analysed with SAP. This section describes the modelling of the following key global behavioural aspects of the terminal structure:

- Static Analysis
- Fatigue Analysis
- Seismic Analysis (used only as a check analysis over IBDAS model results)

The following elements have been designed and verified with the global model:

- Steel box
- Transverse steel truss

- Longitudinal webs
- Transverse webs

6.5 Local Behaviour

The global modelling and design verifications described throughout this report is supplemented with semi-local and local shell elements modelling of the following components:

- Concrete deck slab
- Predalles
- Fatigue analysis

6.5.1 SAP Local Shell Model

The local FE-models are also performed in the computer program SAP 2000. In total four different models have been analysed:

- Concrete predalles simply supported beam model.
- Concrete slab FE-local model to determine the stress concentration in roadway and railway.
- HE500B transversal top beam model (treated in the previous FE-local model).
- FE-local model of a transversal section to determinate fatigue stresses.

The general purpose of the local models is to verify the proposed element solution and to document that the stress flow is acceptable with respect to ULS load combination.

To ease the calculation the model will be composed by shell elements whereas the transversal stiffeners and the transversal beams will be modelled by beams elements. In order to determine the most accurate stress distribution all the other elements will be modelled as shell elements. The implementation of beam elements is meant to reduce significantly the calculation time without affecting the accuracy of the analysis.

7 Design Principles

This section describes the methods used to verify the terminal structure components at SLS, ULS, and FLS.

7.1 Serviceability Limit States

This section describes the methods used to verify performance of the terminal structure structural components at SLS.

7.1.1 Steel deck Elements

Longitudinal elements capacity is governed by first yield point at any cross section as there is insufficient ductility available for plastic behaviour. The section behaviour and capacity is the same for all load combinations, with the exception of potential reductions for shear lag under SLS loading. However, shear lag does not affect the deck elements section properties because of the long lengths over which bending moments develop and therefore ULS and SILS load combinations govern the terminal structure deck design. Verification for SLS load combinations is not necessary for stress.

Displacement or deformations that could limit the use of the structure under SLS load combination will be calculated.

Verification of the longitudinal elements for ULS is described in Section 7.2

The transverse diaphragms and stiffeners are used to provide restraint against out-of-plane buckling of the longitudinally stiffened panels. The full plate diaphragms and the trusses are used to prevent excessive torsional distortions in the members.

Similar to the longitudinal elements, verification of these components for SLS load combinations is not necessary. Verification of the transverse elements for ULS is described in Section 7.2

7.1.2 Concrete deck Elements

The concrete slab is made by a concrete cast upon a concrete predalles; the effects due the dead load of the concrete cast are all taken by the concrete predalles. All the effects due variable man-generated actions as the traffic loads are supported by the concrete slab for its complete thickness.

SLS loading during the casting stage will be studied for the predalles and cracking will be studied for the concrete slab. The crack limit is assumed w = 0.3 mm for SLS Frequent.

7.2 Ultimate Limit States

Verification of the terminal structures components for the ULS, in general, follows the same procedures. The only difference in the verifications for the two limit states is that the material partial factors are assumed equal to 1.0 for the SILS verifications.

7.2.1 Steel element verifications

The steel elements that compose the entire deck of the terminal structure have been verified for the following cases:

- Resistance of cross-sections to general stresses,
- Resistance of members to instability assessed by member checks,
- Resistance of plated structures subjected to out of plane loading to instability.

7.2.2 Resistance of cross-sections:

The steel elements are loaded by several actions acting simultaneously in different directions.
The verifications must be done according to the cross-section class that depends on the type of stress state acting on the cross section. The classification is made according to the Italian standard depending on the type of cross-section and its rotation capacity. Four classes are identified and the type of analysis depends on the class of the cross-section. For the Class 4 cross-section the analysis is always elastic.
All around the cross-sections have result to be in Class 4 so for those elements the verifications are undertaken whilst considering an elastic behavior. For those elements which have been classified differently from class 4 an elastic verification has been made also.
To compose the various actions considering an elastic material the Von Mises's criterion has been used including safety partial factors for the materials strength:

$$\sigma_{x,Ed}^2 + \sigma_{z,Ed}^2 - \sigma_{z,Ed}\,\sigma_{x,Ed} + 3\,\tau_{Ed}^2 \leq (f_{yk} / \gamma_{M0})^2$$

Where:

$\sigma_{x,Ed}$ is the local longitudinal stress at the point of consideration,

$\sigma_{z,Ed}$ is the local transverse stress at the point of consideration,

τ_{Ed} is the local shears stress at the point of consideration,

γ_{M0} is the material strength safety factors.

7.2.3 Classification of cross-sections

Each cross-section has been classified considering its rotation capability and four classes are defined. The classification can be made referring to tables provided b the Italian standards NTC 2008.
If the cross-section is made up with two or more steel plates as in the case of the steel elements of the box-girder the class assumed for the entire cross-section in the greatest of the classes of each elements.
The steel elements considered are divided in three principal elements: web, flanges and ribs.
The classification of each element and of each cross-section is provided in the verifications report.

7.2.4 Resistance of members to instability

The elements loaded by negative axial forces (compression) must be verified against buckling as imposed in the Italian instructions NTC 2008.
The elements most subjected to compression are the diagonal steel beams HE300 B, which constitute the transverse truss.

A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1$$

where:

N_{Ed} is the axial design force,

$N_{b,Rd}$ is the design buckling resistance of compression member

$$N_{b,Rd} = \frac{\chi\, A\, f_{yk}}{\gamma_{M1}}$$ $$N_{b,Rd} = \frac{\chi\, A\, f_{yk}}{\gamma_{M1}}$$ for Class 1,2 and 3 cross-sections

$$N_{b,Rd} = \frac{\chi\, A_{eff}\, f_{yk}}{\gamma_{M1}}$$ for Class 4 cross-sections

The reduction factor χ depends on the cross-section type and steel strength; it's determined considering the appropriate non-dimensional slenderness λ according to:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}$$

Where:

$$\Phi = 0.5\,[1 + \alpha\,(\bar{\lambda} - 0.2) + \bar{\lambda}^2)]$$

$\bar{\lambda}$ is the non/dimensional slenderness so determined:

$$\bar{\lambda} = \sqrt{\frac{A\, f_{yk}}{N_{cr}}}$$ for Class 1,2 and 3 cross-sections

$$\bar{\lambda} = \sqrt{\frac{A_{eff}\, f_{yk}}{N_{cr}}}$$ for Class 4 cross-sections

$$\bar{\lambda} = \sqrt{\frac{A_{eff}\, f_{yk}}{N_{cr}}}$$

α is an imperfection factor corresponding to the appropriate buckling curve and obtained from table 4.2.VI find in NTC 2008 depending on the cross/section, the type of steel and the bending plane.

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross sectional properties and on the elements length, obtained applying the classic elastic Euler theory. For the first mode:

$$N_{cr} = \frac{\pi E I}{{L_0}^2}$$

where:

L_0 = buckling length in the bulking plane considered and calculated as = β · L

β = depends on the restrain conditions of the element in the buckling plane considered.

The buckling effects may be ignored if $\overline{\lambda}$ < 0,2 or Ned < 0,04 N_{cr} .

Is the slenderness λ < 250 for secondary elements and λ < 200 for principal elements:

$$\lambda = \frac{L_0}{i}$$

where:

L_0 = buckling length in the bulking plane considered and calculated as = β · L

β = depends on the restrain conditions of the element in the buckling plane considered.

I = radius of gyration about the relevant axis

7.2.5 Resistance of plated structures subjected to out of plane loading to instability

According to Italian standards the steel plated structures that constitute the box girder must be verified for buckling collapse and the rules for structural design and subsequent verifications of stiffened plates are provided below.

For un-stiffened plates the buckling verifications must be done if the relation below is not satisfied:

$$\frac{h_w}{t} > \frac{72}{\eta} \cdot \sqrt{\frac{235}{f_{yk}}}$$

In this case this check has no sense considering that the plates are longitudinally and transversally strengthened with open stiffeners.

Therefore the following relation to take into account for the buckling verifications is:

$$\frac{h_w}{t} \geq \frac{31}{\eta} \cdot \varepsilon \cdot \sqrt{k_\tau}$$

Where:

t = panel thickness

h_w= Panel height

ε = $(235/f_y)^{½}$

η =1.2

kτ = minimum buckling coefficient for shear actions.

If this relation is not satisfied the Italian instructions NTC2008 contains rules to be applied for the verifications of stiffened plated against bucking collapse.

The verifications made and reported in verifications report show all the terms and the criterion followed to make the verifications.

7.2.6 Resistance to shear

This section gives rules for shear resistance of plates considering shear buckling at the ULS.

Shear resistance is calculated according to the requirements of NTC 08, section 4.2.4.1.2.

For unstiffened or stiffened webs the design resistance for shear should be taken as:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta \cdot f_{yw} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}}$$

where: h_w is the panel depth,

k_t is the shear buckling coefficient,

$f\gamma_w$ is the panel yield strength,

γ_{M1} is the material partial factor.

in which the contribution from the web is given by:

$$V_{bw,Rd} = \frac{\chi_w \cdot f_{yw} \cdot h_w \cdot t}{\sqrt{3} \cdot \gamma_{M1}}$$

and the contribution from the flanges is according to the following:

$$V_{bf,Rd} = \frac{b_f \cdot t_f^2 \cdot f_{yf}}{a\left(0.25 + \frac{1.6 \cdot b_f \cdot t_f^2 \cdot f_{yf}}{t \cdot h_w^2 \cdot f_{yw}}\right)\gamma_{M1}} \cdot \left[1 - \left(\frac{M_{Ed}}{M_{f,red}}\right)\right]$$

where:

$$M_{f,red} = \frac{M_{fk}}{\gamma_{M0}} \cdot \left(1 - \frac{N_{Ed} \cdot \gamma_{M0}}{(A_{fi} + A_{fs}) \cdot f_{yf}}\right)$$

The verification is performed as follow:

$\eta_3 = V_{Ed} / V_{b,Rd} \leq 1.0$

where V_{Ed} is the shear force including shear from torque.

7.2.7 Fatigue assessment

The safety checks of the parts below the zone interested by the rail loads (rail-way) are made according to the Italian railways instructions for fatigue loads and safety checks [RFI 44 F: RFI DTC-ICI-PO SP INF 003 A] . The safety checks of the parts underlying the road ways, that are loaded only by road traffic loads are instead performed accordingly to the Italian code NT08.

As specified by the design basis, the fatigue safety checks are done for an unlimited life of all the components analyzed. The loads and the safety factors required for this type of fatigue checks are used.

The fatigue checks concern only those parts of the steel deck that are most affected by the man-generated loading, as the road and rails traffic.

The most affected parts to the fatigue loads are the welding between the ribs and the panels.

Some of these elements are subjected to both global and local fatigue effects, other elements are mainly subjected by local fatigue effects and for this reason these effects have been determinate using adequate local models.

The parts analyzed for the fatigue effects need to be identified with a detail class in order to make the safety checks, according to NTC 08. These parts are:

- The welding on the cruciform joint between the top transversal steel element HE500B and the transversal rib of the vertical panel made with a ½ HE 600A
- The welding on the cruciform joint between the transversal rib of the vertical panel made by a ½ HE 600A and the transversal rib of the bottom panel made with a ½ IPE 600.
- The welding of the transversal rib of the vertical panel made by an ½ HE 600A and the panel itself.
- The welding of the transversal rib of the bottom panel made by an ½ IPE 600 and the panel itself.
- The welding between the vertical panel and the ½ IPE 600 as the detail 8 of section B-B of the table CG1002 P BX D P SV S8 VS 00 00 00 01 B.
- The welding on the corner between the bottom panel with ½ IPE 600 and the vertical panel with a plate 20 thk.
- The welding on the bottom panel and the plate 180 x 20 thk

The unlimited life safety checks has been made, accordingly to the above described principles, verifying that the maximum stress range defined as $\Delta\sigma=(\sigma_{max}-\sigma_{min})$ are minor then the admitted stress range for the verified details.

The category assigned at each detail indicates the maximum stress-range admitted for that detail in the unlimited life safety checks. Fatigue categories are used according to NTC 08 (reference is made to Figure 7-1, 7-2, 7-3 and Table 7.1).

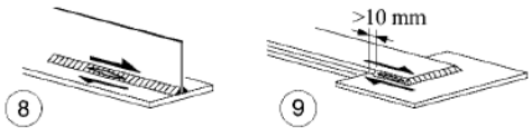
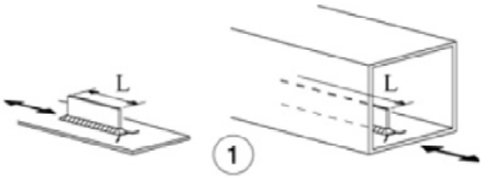
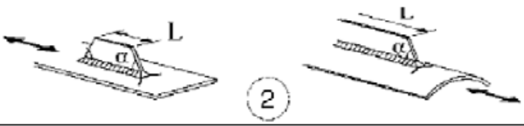
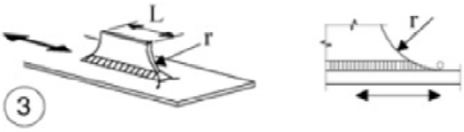
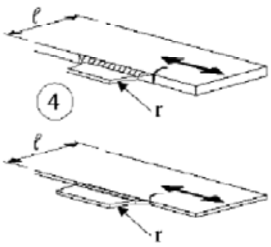
Classe del dettaglio	Dettaglio costruttivo	Descrizione	Requisiti
80		8) Cordoni d'angolo continui soggetti a sforzi di sconnessione, quali quelli di composizione tra anima e piattabanda in travi composte saldate 9) Giunzioni a sovrapposizione a cordoni d'angolo soggette a tensioni tangenziali	8) $\Delta\tau$ deve essere calcolato in riferimento alla sezione di gola del cordone 9) $\Delta\tau$ deve essere calcolato in riferimento alla sezione di gola del cordone, considerando la lunghezza totale del cordone, che deve terminare a più di 10 mm dal bordo della piastra

Figure 7-1: Table C4.2.XVII.b Load carrying welded joints ($\Delta\sigma$)

Classe del dettaglio	Dettaglio costruttivo	Descrizione	Requisiti
80 (a) 71 (b) 63 (c) 56 (d)		Attacchi saldati longitudinali 1) La classe del dettaglio dipende dalla lunghezza dell'attacco (a) L≤50 mm (b) 50<L≤80 mm (c) 80<L≤100 mm (d) L> mm	Spessore dell'attacco minore della sua altezza. In caso contrario vedi dettagli 5 e 6
71		2) Attacchi saldati longitudinali a piatti o tubi con L>100 m e α<45°	
80		3) Fazzoletti d'attacco saldati a piatti o tubi con cordoni d'angolo longitudinali e dotati di raccordo di transizione terminale di raggio r. La parte terminale dei cordoni deve essere rinforzata, cioè a piena penetrazione, per una lunghezza maggiore di r. r≥150 mm	Raccordo di transizione di raggio r realizzato con taglio meccanico o a gas realizzato prima della saldatura del fazzoletto. Al termine della saldatura , la parte terminale deve essere molata in direzione della freccia per eliminare completamente la punta della saldatura
90 (a) 71 (b) 50 (c)		4) Fazzoletti d'attacco saldati a un lato di un piatto o della piattabanda di una trave e dotati di raccordo di transizione di raggio r. La lunghezza L deve essere valutata come per i dettagli 1), 2) e 3). La stessa classificazione può essere adottata anche per piattabande saldate dotate di raccordo di transizione di raggio r. (a) r≥L/3 o r>150 mm (b) L/3>r≥L/6 (c) r<L/6	Raccordo di transizione di raggio r realizzato con taglio meccanico o a gas realizzato prima della saldatura del fazzoletto. Al termine della saldatura , la parte terminale deve essere molata in direzione della freccia per eliminare completamente la punta della saldatura
40		5) Come saldato, senza raccordo di transizione	

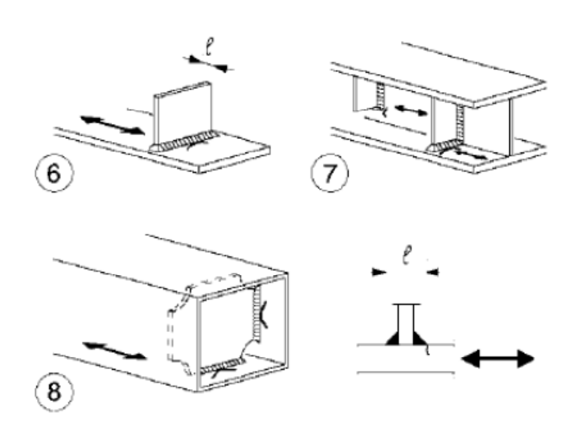
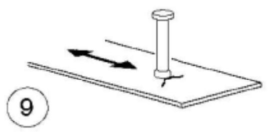
80 (a) 71 (b)		Attacchi trasversali 6) Saldati a una piastra 7) Nervature verticali saldate a un profilo o a una trave composta 8) Diagrammi di travi a cassone composte, saldati all'anima o alla piattabanda (a) ℓ≤50 mm (b) 50<ℓ≤80 mm Le classi sono valide anche per nervature anulari	6) e 7) Le parti terminali delle saldature devono essere molate accuratamente per eliminare tutte le rientranze presenti 7) Se la nervatura termina nell'anima, Δσ deve essere calcolato usando le tensioni principali
80		9) Effetto della saldatura del piolo sul materiale base della piastra	

Figure 7-2: Table C4.2.XVI Weld attachments and stiffeners (Δσ)

Element	Constructional detail	Table	Detail category
1	1 (d)	C4.2.XVI	56
2	1 (d)	C4.2.XVI	56
2b	1 (d)	C4.2.XVI	56
3	8	C4.2.XVII.b	80
4	8	C4.2.XVII.b	80
5	1 (d)	C4.2.XVI	56
6	1 (d)	C4.2.XVI	56
7	1 (d)	C4.2.XVI	56
8	8	C4.2.XVI	80

Table 7-1: Element and detail category

The safety factor γ_M applied for the safety checks is different between the road and rail box girder. This factor amplifies the stress-range computed by the model.

For the evaluation of the fatigue effect, in both road and rail girder and for all elements, the Safe life assessment method is considered; furthermore is assumed that the consequences of the failure due fatigue effects are low. The resulting safety factors γ_M are:

Box Girder	γ_{Mf}
Road	1,15
Rail	1,35

Table 7-2: Partial safety factors

7.3 Deck assessment

7.3.1 Deck: steel elements verification

The strength verifications have been made according to the semi probabilistic method principles. This criteria considers the cross-section strength as function of the cross-section class.

The cross-sections considered are generally in Class 4, which is considered standard for beam realized with welded panels. In this case (Class 4) the geometric proprieties are related to the effective cross-section, which is composed whilst considering reduction factors for every single panel related to local buckling effects.

Taking into account those considerations, the strength verifications are made according to elastic resistance. The verifications are satisfied if the applied stresses are less than the elastic resistance of the cross-sections.

The classification of composite steel-concrete cross-sections is made according to the indications provided for the steel sections, as present in the paragraph 4.3.2.1 of the NTC2008.

The design of panels subject to out of plane loading is conducted, as aforementioned, considering the effective cross-sections property. To determinate the effective cross-section properties it is necessary to define the reduction factors defined in relation to the stresses acting at the ends of the panel.

7.3.1.1 Effective cross-section property definition

After determining the stresses acting on the elements, the next step is to define the effective cross-section of the panels that constitutes steel box considering what specified in C.M. 02.02.2009 n.

61, for whom, is necessary to take into account also the global effects related to the plates and columns.

7.3.1.2 Verification Section

Verification sections have been identified from the analysis of the FE model's results of the shell elements stress state. First is the most critical load combinations identified, then for each of them the stress diagrams on the FE model's elements have been analyzed. The analysis of the stress diagrams permits to identify the most stressed zone.

Verifications have been performed only at the most stressed areas. The most stressed zone, regarding steel elements strength verifications, are the zone nearby the bearings.

The verification sections are reported as stress maps in the design reports for main elements, CG1002-P-CL-D-P-SV-S8-00-00-00-00-01 Design Report - Main elements.

Examples of the above described maps are reported in figures below:

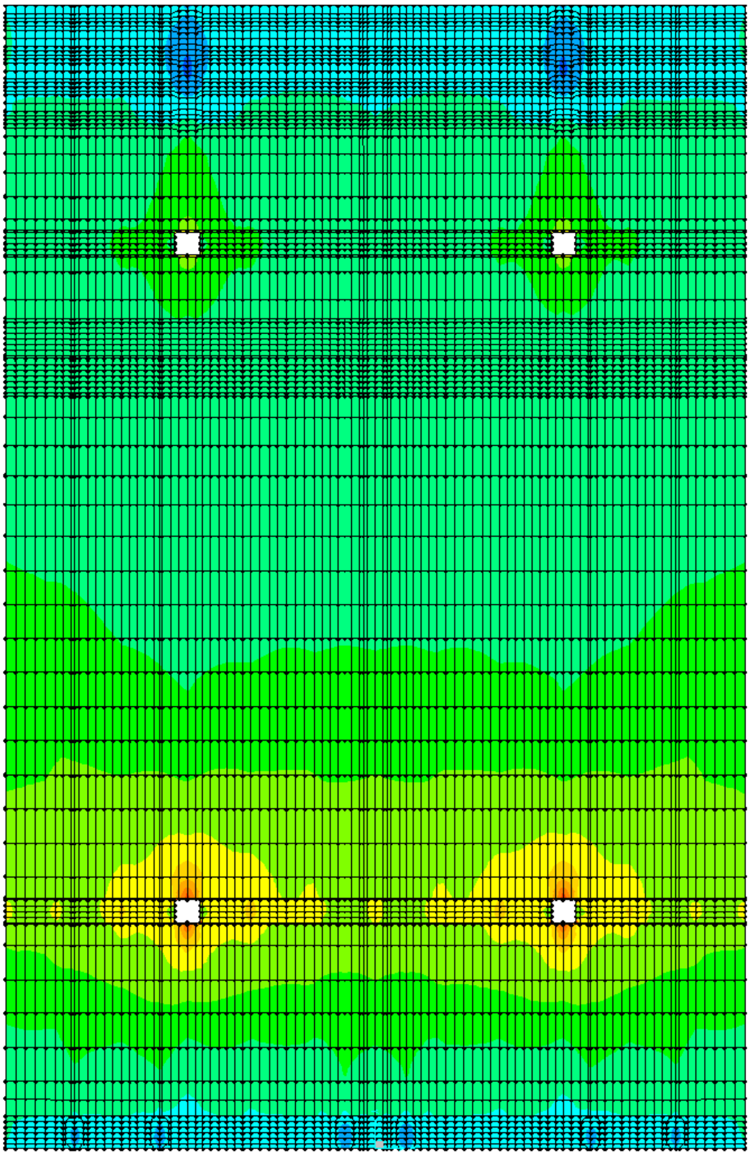


Figure 7-3: Stress of deck - Top view

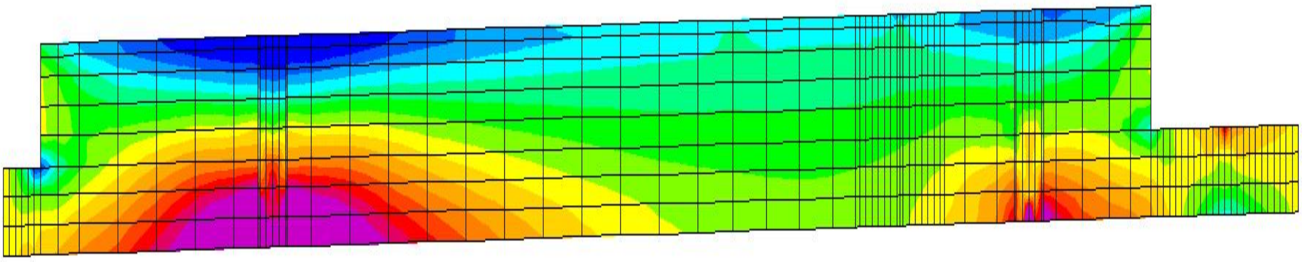


Figure 7-4: Stress map of longitudinal web

7.3.1.3 Verification Spread Sheets - Longitudinal and Transverse Steel Sections

In order to structure and generalise calculations of the longitudinal steel deck sections in terminal structures, a Steel Verification Spreadsheet has been developed based on the NTC08. The verification of the longitudinal steel girders are section verification, stress and buckling checks. The design is made in accordance with the project specific Design Basis and with background in the NTC08.

The overall verification of the suspended deck is done based on the derived section forces obtained from the Semi-local SAP FE-model.

The verification spreadsheet is developed according to the guidelines stated in the NTC08 for steel structures. The purpose made spreadsheet is designed to be used on a closed steel box with any type of outer geometry and location of longitudinal stiffeners. The spreadsheet calculates gross and effective section properties for a given set of parameters including the section geometry, plate thicknesses, stiffener types and spacing, diaphragm spacing and length of the moment diagram. The calculations are based on sectional forces from SAP FE- model.

7.3.2 Deck: concrete elements verification

The concrete deck has been studied with local FE-models, performed in the computer program SAP 2000. The following models have been analysed:

- Concrete predalles simply supported beam model.
- Concrete slab FE-local model to determine the stress concentration in roadway and railway.

7.3.2.1 Concrete verification

The verifications 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 crack width (Ap.Fess.) with the allowable crack width.

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

7.3.3 Deck: deflection verification

For all structure configurations loaded with the classified characteristic vertical loading in accordance with EN 1991-2, 6.3.2 (and where required classified SW/0 and SW/2 in accordance with EN 1991-2, 6.3.3) the maximum total vertical deflection measured along any track due to rail traffic actions should not exceed L/600.

In the middle span of the Terminal foundation we find the following values:

Terminal structure	Max. Calculated deflection	Max. Allowable deflection
SICILIA	0.0456 m	0.0917 m
CALABRIA	0.0287 m	0.0583 m

Table 7-3: Deck deflection

8 Articulation

8.1 Introduction

The Messina Bridge is a very large and flexible bridge and therefore the loads will result in very large movements of the suspended structure. Transverse wind load on the structure results in up to 12.5 m horizontal movement of the bridge girder, traffic loads on the bridge results in vertical movements of the bridge girder up to 4,1 m and movements in the longitudinal direction up to 6.7 m when combined with wind loads in SLS II. Large movements of the bridge girder results in pronounced wear for the mechanical components, it is therefore important to reduce the movements as much as possible in order to obtain a cost effective design and reduce interference of the traffic due to repair and maintenance as much as possible.

The terminal structures are affected by the same requirements needed for the bridge and by the choices made in terms of articulation systems for the bridge, and for the viaducts landing on them.

It is furthermore required that the comfort is high for the users of the bridge when passing the bridge by road or rail.

The mechanical components such as bearings and expansion joints for the bridge should as far as possible remain within the range which has proven feasible for other bridges.

The bridge and the terminal structures should be able to survive severe earthquakes and for this purpose it is an advantage if the bridge is flexible in order to limit the forces in the structure.

The objective is to introduce a system of devices that prevents movement at normal operation conditions but during severe load conditions such as earthquake will allow movements of the girder and will be able to dissipate energy.

The components of the devised system are introduced in the following sections.

8.2 Scope

The design of the hydraulic buffers, bearing and expansion joints are proposed by the manufacturers based on technical specifications provided by COWI.

The technical specifications are in accordance with:

- CG.10.00-P-RG-D-P-GE-00-00-00-00-00-02, "Application Manual to Design Basis, Structural, Annex"
- GCG.F.05.03 "Design Development – Requirements and Guidelines," Stretto di Messina, 2004 October 22.

8.3 Articulation

The main principles for the Articulation System has been changed compared to the Tender Design. Hydraulic buffers in the longitudinal directions have been foreseen in accordance with the load/displacement characteristics included in the Tender Design. Hydraulic buffers in the transverse directions have been instead eliminated.

The below reported drawing gives an overview of the terminal structure articulation:

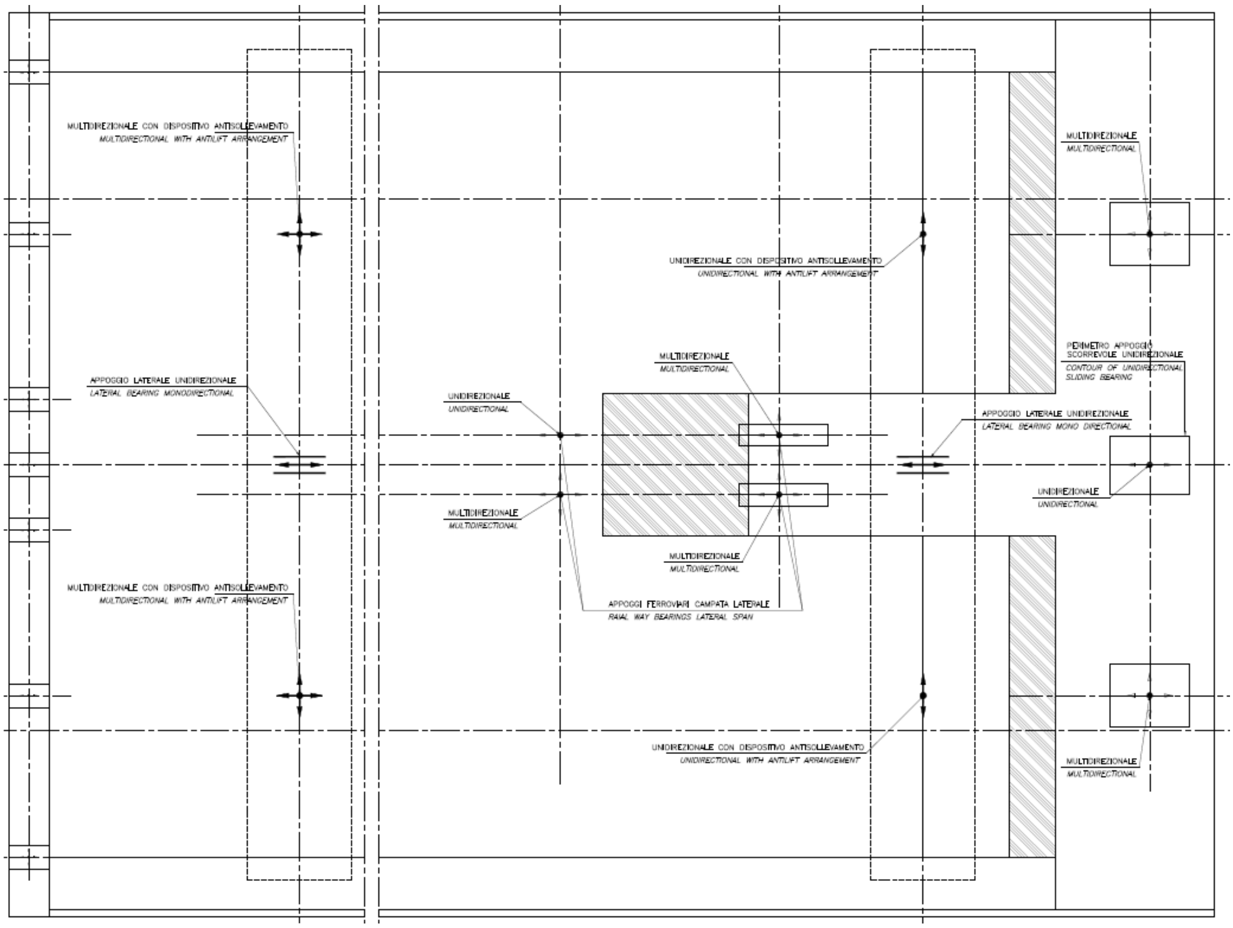


Figure 8-1: Terminal structure articulation

A transversal restraint will be installed at the transverse beam of the piers, to block transversal movements between substructure and superstructure.

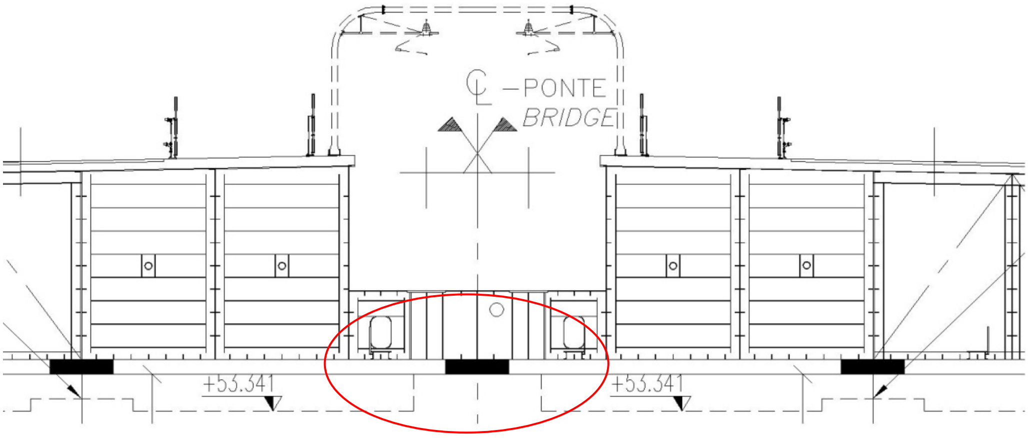


Figure 8-2: Monodirectional lateral bearing, transversal movements not allowed

The expansion joints in the roadway proposed by MAGEBA have been modified. They are now wider, both for roadway and railway. Furthermore on the roadway deck an additional space of approximately 1.2 m in vertical is required for these devices.

An additional zone for temporary jacking around the bearings between piers and superstructure has been provided in order to allow maintenance and replacement of the devices.

8.3.1 Bearings

The terminal superstructures are supported by bearings placed on the piers. At both ends of the suspended bridge structure the railway girder is protruding into the terminal structure and bearings for vertical loads are located at the end of the railway girder at cross girder type T8 and at cross girder type T7. Two further bearings for vertical loads are found at the crossbeam connecting the rail and road girders at the terminal structure.

In the transverse direction the railway and road girder is guided by bearings A11 and A8:

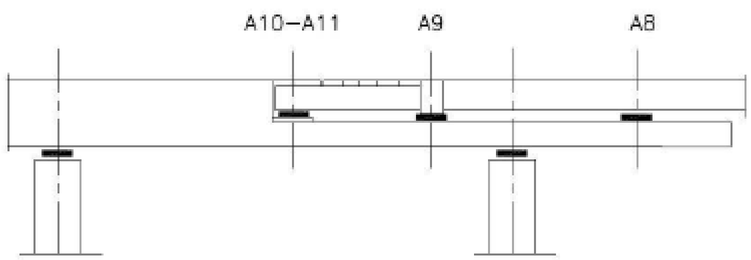


Figure 8-3: Bearing layout at terminal structure on Sicilia side - side view

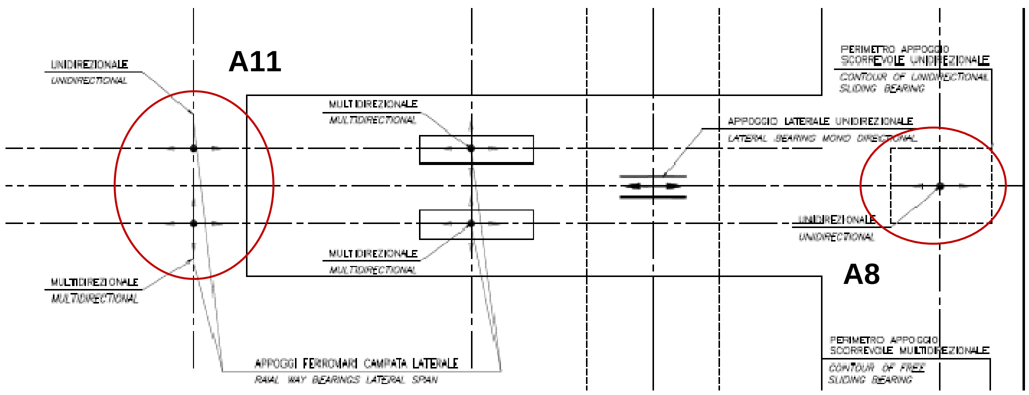


Figure 8-4: Bearing layout at terminal structure on Sicilia side

8.3.2 Antiuplift devices

Due to the presence of uplift forces in the bearings, bearings will be of fixed type and equipped with antiuplift restraints.

The figure below shows the scheme of these devices which will be further developed during the next design phase:

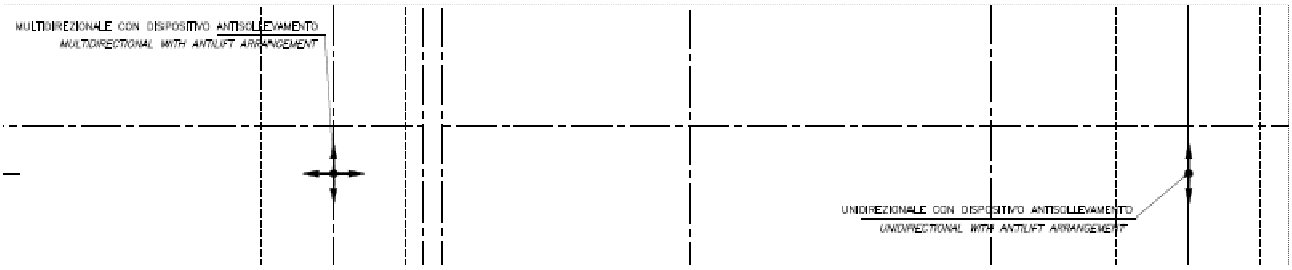


Figure 8-5: Antiuplift devices on one side

8.3.3 Expansion Joints

Road expansion joints and rail expansion joints are installed at the terminal structures.

8.3.3.1 Roadway expansion joint

The roadway expansion joints are of a modular type. The roadway expansion joints at the extremities of the terminal structure are adapted with a joist box to facilitate the large structural movement at earthquakes.

8.3.3.2 Railway expansion joints

The railway expansion joints are longitudinally moveable stock rails type which facilitates large longitudinal and rotational movements.

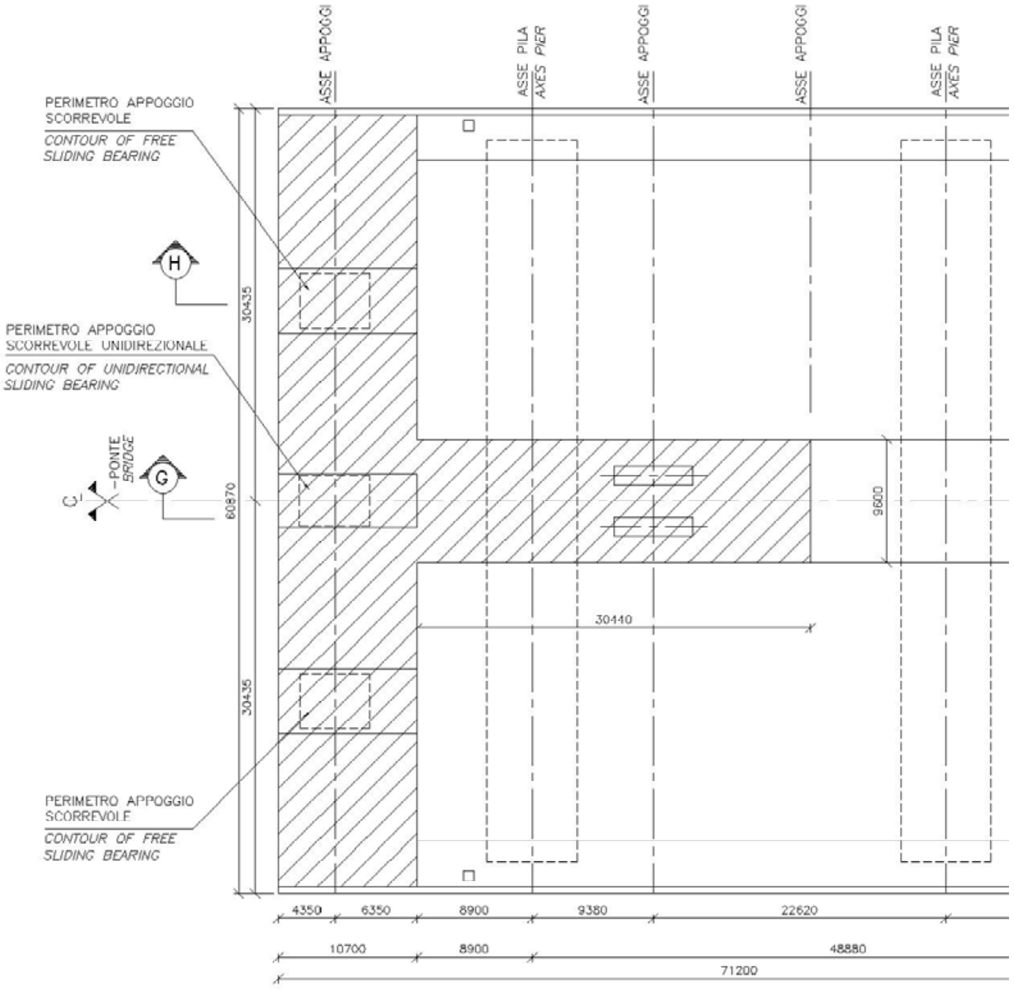


Figure 8-6: Road and railway expansion joints zone on terminal structures sea side