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)
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INDICE

0. Preface

The Contract document GCG.F.04.01 has been updated to account for the developments of the Tender Design, the interpretative clarifications occurred in the tender stage and during the beginning of Progetto Definitivo and the design modifications approved and/or decided by the Client. These modifications have particularly concerned:

- the increase of the height of the tower from 382 m to 399 m above mean sea level.
- the inversion of the of the roadway travelling direction on the bridge
- the transposition of the recent norms and provisions introduced by D.M.14.011.2008 *"Norme Tecniche per le Costruzioni"*, (in the following indicated as "NTC 08") and of the new RFI "Instructions" on railway bridges, both developed in accordance with the Eurocodes.

NTC 08 are transposed, as agreed with the Client, accounting for the complex interactions between the norms and the contract performance specifications introduced in GCG.F.04.01 that have been designed and calibrated according to the special and exceptional nature of the bridge. NTC 08 have been fully implemented for what concerns fatigue design (Sec. 4.2), local design of the structural system (Sec. 5 and 6), as well as the characterization of the soils and the safety of the foundation works (Sec. 7), except for the application of the seismic specification introduced by GCG.F.04.01.

It is kept the body of the original document GCG.F.04.01 rev. 0, with the same chapters heading and numbering.

1. Introduction

This document defines the basis of design and establishes the expected performance levels for the Strait of Messina Main Bridge (later on also denominated the Bridge, or the structure) which shall be achieved and satisfied in all subsequent design development, in the construction phases and by the completed and tested Bridge structure.

This document has been prepared in compliance with the Progetto Preliminare as approved by the Interministerial Committee for Economic Planning (CIPE) on 1 August 2003, which defines the geometrical properties and the mandatory performance requirements that must be satisfied by any alternative design proposal submitted by the Tenderers/General Contractor for the Bridge. This design has been developed on the basis of the document “Indirizzi Progettuali e Deliberazioni per il Progetto Preliminare” ("Design Approaches and Decisions for the Progetto Preliminare") and the vote of the Consiglio Superiore dei Lavori Pubblici (Public Works and Infrastructure Ministry) in the General Meeting No. 220/97 on 10/10/1997.

The Progetto Preliminare in particular specifies:

- the geographical coordinates of the centroids of the support sections of the towers,
- the geographical coordinates of the cable anchor blocks, and the geographical alignment of the Bridge centreline,
- the height of the towers,
- the length of the deck and the coordinates of its support points,
- the width and functional composition of the deck,
- the clearance of the Bridge soffit above sea level,
- the distance between hangers, and
- the number of main cables.

The expected safety and performance of the structure shall be achieved, with a safe margin and cost optimisation, both during the construction and structural testing stages and during the design[1] life of the structure, taking into account all the chemical-physical phenomena including corrosion and

[1] According to the definitions from NTC 08 chapter 2.4.1, both nominal life and design life of the Bridge are taken equal to 200 years, in relation to the seismic response and in compliance with the previous development of the present document. Individual components may have their own “service life” lower than 200 years, as specified in Section 4.2 and in Section 1.6 of document GCG.F.05.03.

fatigue, that can give rise to the degradation of the mechanical properties of materials, structural components and of structures.

This document defines the loading actions that must be taken into account in the assessment and analysis of the stresses in the Bridge, distinguishing between environmental actions, caused by natural phenomena (wind, earthquake, temperature actions), and man-generated actions. For environmental actions, determined on the basis of statistical sets of measurements taken over long periods, the numerical parameters and correlations describing the natural phenomena explicitly considered are provided. These actions and the criteria for their application to the structure are at the basis of the design and represent mandatory norms of reference for the Tender Design.

During the design and construction stages, the General Contractor shall extend the statistical populations of the measurements and check the correlations describing these phenomena, and without delay shall inform and submit appropriate documentation in this regard to the Client.

The reliability of the structure, as defined and prescribed in this document in terms of safety and functionality, shall not be reduced without explicit authorisation from the Client.

The methodologies and the design solutions proposed by the General Contractor will be assessed, at each level, on the basis of their suitability, effectiveness, simplicity, robustness and reliability.

This document also addresses the basic materials to be used, the adoption of devices to control the structural behaviour of the Bridge, and the structural and environmental monitoring system.

The occurrence of changes to the global and local geometry of the Bridge structure, and to the stresses, strains, deformation, and displacements it experiences shall be monitored in real time, so as to allow the management plan to be implemented, even in possible emergency situations. At the same time, the respect of the performance levels expected of the structure in use shall be checked in real time, so as to allow the current functionality of the structure to be determined and the adopted management plan to be carried out.

With regard to the loading actions on the Bridge, the assessment of the strength of the structural components, the safety of the Bridge and its behaviour in operating conditions, reference shall be made to Italian standards as listed in section 12 References, within their range of application as defined therein. Outside the range of application of the Italian standards, the recommendations contained herein shall be applied.

These design criteria are applicable and mandatory for the main Bridge structure, but not for the other ordinary works completing the road and rail link between Sicily and Calabria which must in any case observe current Italian Regulations as in force at the Progetto Definitivo time (1 April 2010) and listed in chapter 12.

Should the General Contractor identify any omissions, inconsistencies or ambiguities in this document, he shall promptly refer them to Stretto di Messina S.p.A. (also denominated as the“Client”) in order to resolve together with the latter any such equivocal issue.

During the designing stage of Progetto Definitivo the General Contractor's proposed design changes will be evaluated and shall only be implemented with explicit authorization from the Client.

2. Description of the bridge and its structural decomposition

Since 1984 a continuous series of design solutions has been developed by Stretto di Messina S.p.A. in collaboration with eminent Italian and foreign researchers, in accordance with the guidelines and requirements of the "Commissione dei Consulenti ANAS" (Commission of ANAS Consultants), the "Delegazione di Alta Sorveglianza delle Ferrovie dello Stato" (Superintendence of the State Railways), the prescriptions of the vote of the Consiglio Superiore dei Lavori Pubblici, General Convention No. 220/27, meeting held on 10/10/1997, and the contents of the document "Indirizzi progettuali e deliberazioni per il Progetto Preliminare" (Design criteria and deliberations for the Progetto Preliminare) by the Comitato Tecnico Scientifico (Technical Scientific Committee) at the Ministry for Transport and Infrastructures, dated 13/12/2002. This activity has led to the preparation of the Progetto Preliminare approved by CIPE. Even after the above approval the studies, analysis and research activities have continued. The study documents included in the

"Studies and informative documents" are based on the results of the above.
The basis of design and the expected serviceability and safety performance levels defined in this document have been checked with reference to the Progetto Preliminare and the above mentioned study documents that describe the analytical validation, achieved by means of extensive numerical simulations, and the experimental validation of the performance levels of the structure under environmental and man generated actions. The Progetto Preliminare and the attached documents therefore represent the reference basis for any improved design proposals that may be presented by the Tenderers in their Tender Design.

The Bridge structure, as defined by the Progetto Preliminare, by the Tender Design and by the agreements for the start of Progetto Definitivo, is composed of (see summary chart in Fig.1):

- Tower foundations: they consist of massive plinths of circular shape under each leg, bearing onto a sub-foundation formed by jet-grouted piles or diaphragms. The plinths are connected by beams of rectangular section;
- Anchor blocks/Cable anchorages: massive reinforced concrete structures, of prismatic shape and trapezoidal section;
- Towers: framed structures in a plane transverse to the Bridge, composed of two legs and three cross beams. They have a total height of 399 m above mean sea level. The steel legs are box sections of losenge shape (octagon). The tower cross beams are located: at the top of the tower, and at two intermediate levels.

- Saddles: placed to support/deviate the main cables and located at the tops of the towers and in anchor blocks;
- Main cables: two for each edge of the deck;
- Hangers: arranged at 30m centres and consisting of groups of steel wire ropes;
- Road box girders: comprising steel box sections stiffened by longitudinal trough stiffeners and transverse diaphragms. Individual box girders connected to supporting cross girders. The box girders are completed by aerodynamic fairings.
- Rail box girder: formed of steel skin plates stiffened by longitudinal stiffeners and transverse diaphragms; in particular, longitudinal structural members are arranged under each track. The railway box girder sections are connected to the supporting cross girders.
- Cross girders: arranged at 30m centres, and supported by the hangers, they support the rail and road box girders; formed of a close box section of variable height.

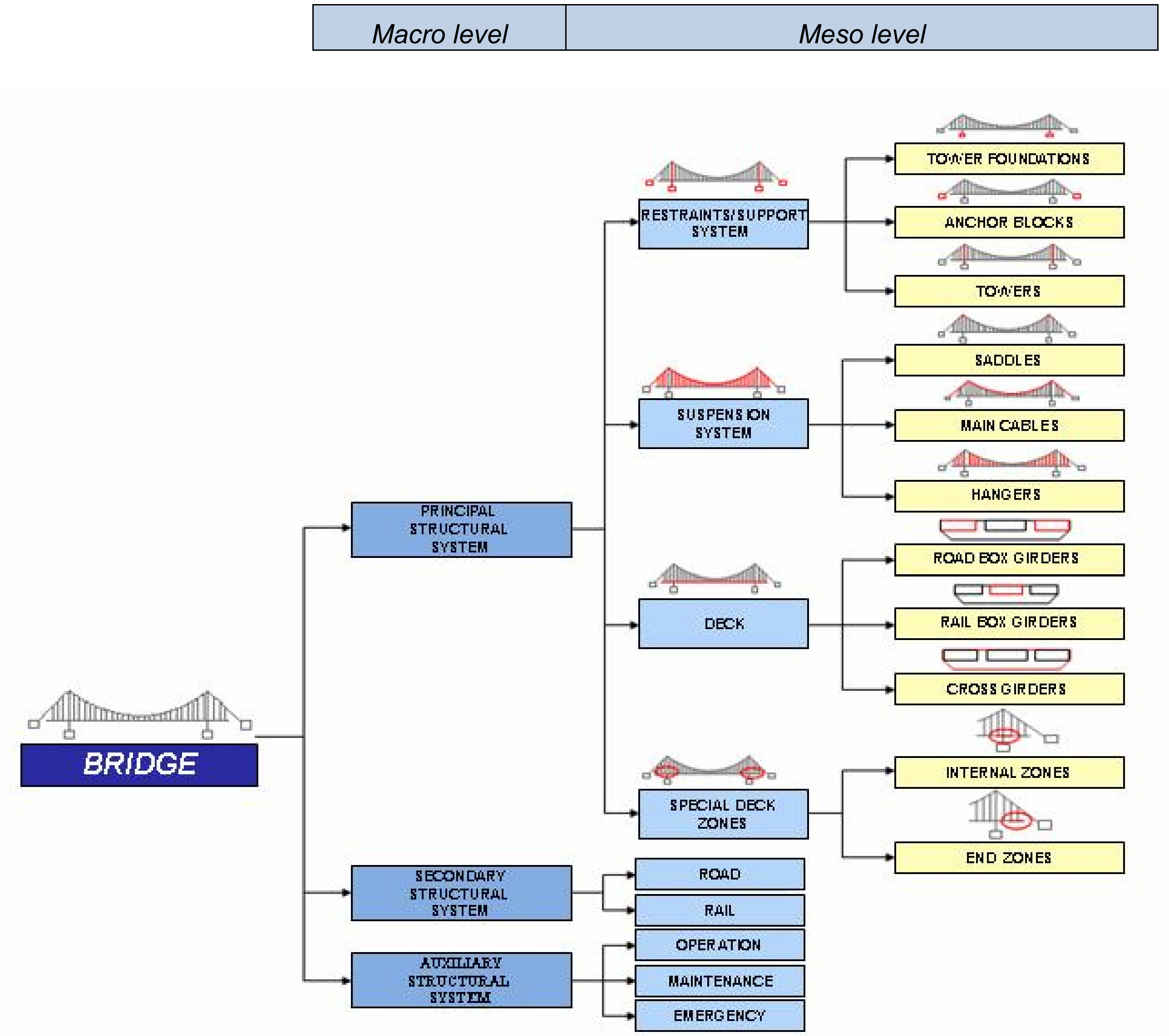
The restraint to lateral movement of the deck is provided by restraint devices located at the towers. The ends of the deck box girders are free to move in the longitudinal direction.

The overall structural system of the Bridge is broken down into a hierarchy of sub-components through the ranking of structural levels presented in Table 1 and Fig.1.

Table 1: Hierarchical decomposition of the overall Bridge system.

Structural level	Structural components
Macro-level	Structural systems of the Bridge (principal, secondary, auxiliary)
Meso-level	Structures and sub-structures
Micro-level	Structural components

Figure 1: Hierarchical decomposition of the Bridge system.



Every single structural part, that once assembled and connected forms a structure or a sub-structure, is defined “structural element” or “structural component”.

To the different structural levels are assigned different levels of reliability, in terms of functionality, durability and safety, and different intensities of the applied loading actions.

With regard to structural failure conditions, this decomposition allows single critical mechanisms to be ranked in order of risk and consequences of the failure mechanism.

With reference to their structural function, the safety required levels and their repairability, structures and sub-structures are distinguished in:

Primary Components (C1), critical, non-repairable or which require the Bridge to be placed out of service for a protracted period in order for them to be repaired;

Secondary Components (C2), repairable, possibly with restrictions on the operation of the Bridge.

The components identified in the structural decomposition (Table 1) are classified as primary and secondary components in Table 2, according to their reparability.

Table 2: Classification of the structural components identified in the structural decomposition of the Bridge

Macro-level	Meso-level		Primary Components (C1)	Secondary Components (C2)
	Structures	**Sub-structures**		
Main[2]	Restraint/support system	Tower foundations	X	
		Anchor blocks	X	
		Towers	X	
	Main suspension system	Main cables	X	
		Saddles	X	
	Secondary suspension system	Hangers system	X	
		Hanger group[3]		X
		Tie down hangers	X	
	Standard deck	Cross girders	X	
		Rail box girder		X
		Road box girder		X
	Special deck zones	End structures and expansion joints		X
		Near towers and restraint systems		X
Secondary	Road			X
	Rail			X
Auxillary	Operation			X
	Maintenance			X
	Emergency			

The hanger system is classified as a main structural component in relation to the global structural safety of the Bridge, whereas an individual hanger group, taken singly, is considered a secondary component due to its repairability and/or replaceability.

[2] It is assumed that "Main" corresponds to "Principal structural system" as indicated in Figure 1.

[3] Hanger group: system of vertical strands that support each deck cross girder at each end.

3. Reliability of the Bridge

Safety, durability and serviceability checks shall be carried out using Limit State Methods in accordance with current Italian Standards listed in section 12 References.
Other methods of analysis may be used, provided they are scientifically established and that the safety levels achieved are not lower.
The Bridge shall be designed and built in such a way as to:

- ensure structural safety and functional quality throughout its design service life (Safety and Serviceability)
- reduce, or at least not amplify, effects due to external disturbances (such as natural environmental or man-generated conditions) or internal disturbances (such as alteration of materials and components and variability due to the manufacturing and assembly processes), also thanks to the intrinsic ductility properties at the material, component and system levels (Structural Robustness)
- pursue a suitable structural configuration that will ensure:
  - easy access for inspection, so that possible deficiencies and defects may be monitored, detected and promptly identified.
  - maintainability and replaceability of the structural elements, via ordinary and extraordinary maintenance works.

The General Contractor may, upon approval of Stretto di Messina S.p.A., address the design issues not explicitly considered herein, also through investigations and procedures based and calibrated on suitable experimental tests (testing for increased reliability levels). The results of such laboratory tests may be used to define safety factors, which shall be in any case subject to the approval of Stretto di Messina S.p.A.
Where the Italian Standards do not fully cover the required checks and issues, the General Contractor shall adopt the Eurocodes or other design codes recognized at international level. Stretto di Messina S.p.A. retains the right to accept the use of such codes and proposals.

3.1 Structural safety and serviceability

The structural safety and serviceability of the Bridge shall be achieved by verifying the performance levels that the Bridge provides during its service life in the various loading conditions (levels) referred to in Table 3.

Table 3: Loading levels and corresponding Limit States

Loading level	Associated Limit State	Acronym	Description
Level 1	Serviceability Limit States	SLS1	• Road and rail runability is guaranteed • No structural Damage • The structure remains in an elastic state and deformations are reversible
		SLS2	• At least rail runability is guaranteed • No structural Damage • The structure remains in an elastic state and deformations are reversible
Level 2	Ultimate Limit States	ULS	• Temporary loss of serviceability allowed • The main structural system maintains its full integrity • Structural damage to secondary components is repairable by means of extraordinary maintenance works
Level 3	Structural Integrity Limit States	SILS	• Complete Loss of serviceability, even protracted in time, is permitted • The survival of the following elements of the structural system must be guaranteed: restraint and support system, main cables, saddles

The checks at the SLS (SLS1 and SLS2) define the functionality of the structure in conditions of normal use. The SLS checks include deformation of deck and tracks also in relation to the issues of comfort and safety of travelling vehicles. Amongst the effects to be checked are deflections, slopes, curvatures, visibility distances, velocities and accelerations related to the Bridge runability and the interaction of the structure with vehicles, as well as the kinematic effects related to the correct functioning of rail and road movement joints. The specified navigation clearance will have to be satisfied as well. The SLS1 and SLS2 checks involve controlling that the stress states created in the structural components remain in the elastic range. The replacement of those structural elements for which replacement is allowed, shall not affect the functionality of the structure.

The checks at the ULS (SLU), in general terms, shall ensure the safety of people and the structure. The ULS checks refer to conditions in which the ultimate capacity of the materials and structural components is reached resulting in loss of functionality of the structure as a consequence of collapse of secondary structural components. As a result extraordinary maintenance operations are required[4].

Due to the exceptional nature of the structure, Structural Integrity Limit States (SILS) shall also be considered, that guarantee the survival of at least the restraint and support system, as well as that of the main cables and the saddles, to extreme accidental and environmental loading conditions. Even in these extreme loading conditions, the robustness of the structure shall be verified, checking the scale of the structural damage is not disproportionate to that of the loading action.

3.2 Design life and return periods

The design life (Ld) of the Bridge shall be assumed to be:

Ld = 200 years[5]

This value is to be used in the design to determine environmental actions, the effects of fatigue and of the degradation of the mechanical properties of materials.

Variable natural/environmental actions (earthquake, wind and temperature loads), characterised in general terms by cyclical trends, described in Par. 5.3, are defined in terms of the various return periods given in Table 4 relating to the defined loading levels (Limit States).

[4] The English translation of F.04.01 has been modified in order to understand the meaning of the sentence. It may not be necessary to change the original Italian text.

[5] The restraint/support system, the main suspension system and the railway deck must possess a design life longer than 200 years. Other components may have shorter design life.

Table 4: Return periods for natural/environmental actions.

Loading level	**Associated Limit State**	**Acronym**	**Return period**
Level 1	Serviceability Limit State	SLS1	50 years
		SLS2	200 years
Level 2	Ultimate Limit States	ULS	2,000 years
Level 3	Structural Integrity Limit States	SILS	According to the contingency scenarios considered

Variable man-generated actions are defined by the nominal values indicated in Par. 5.2.

Permanent actions are calculated on the basis of the geometrical dimensions and specific weights of the various structural and non structural components of the Bridge.

From the actions defined in Chapter 5, the loading conditions are described by the scenarios of Chapter 6.

4. Expected performance levels

The structure shall satisfy the following performance criteria listed in order of importance:

1. Performance for structural safety
2. Performance for serviceability
3. Performance for durability;

Under extreme situations (SLIS), the survival of the following primary elements of the main structural system (as defined above in Table 2) is required:

- restraint and support system,
- main cables and saddles (main structural elements of the suspension system).

4.1 Performance relating to structural safety

In general terms, in relation to safety aspects, the Bridge may be subject to the damage levels indicated in Table 5.

The various structural components identified in the hierarchical decomposition of the Bridge have associated damage levels corresponding to Serviceability Limit States (SLS1, SLS2), Ultimate Limit States (ULS) and Structural Integrity Limit States (SILS), as shown in Table 6 below.

Table 5: Definition of damage levels.

Damage level		Abbreviation	Description
1	No damage	ND	All structural elements and restraint systems retain their nominal performance capacities, remain in elastic state, and do not present any significant degradation due to fatigue.
2	Degradation damage	DD	Degradation of the mechanical properties of the materials after an appropriate period of service due to environmental actions (corrosion) or cyclical actions (fatigue & wear). These effects shall be allowed for in the sizing of structural sections and shall be eliminated or minimized through scheduled maintenance operations.
3	Minimal damage	MD	Occurrence of localised slight inelastic behaviour which does not alter the overall performance capacities of the Bridge. This can be made good by means of ordinary maintenance operations, in any case guaranteeing the road and rail traffic .
4	Repairable damage	RD	Occurrence of localised inelastic behaviour which alters the overall performance capacities of the Bridge. This can be made good through extraordinary maintenance operations, involving partial and temporary closures of the Bridge.
5	Significant damage	SD	Occurrence of inelastic behaviour which significantly alters the overall performance capacities of the Bridge. It corresponds to a serious damage to the structure which may require the reconstruction of entire structural components. The damage can be made good by significant extraordinary maintenance operations, which may involve prolonged closures of the Bridge.

In the following Table 6, with reference to the last 4 columns, the allowable damage levels shall be interpreted as follows[6]:

- The damage levels indicated in the leftmost column (1st) are acceptable for all levels of

[6] Source: clarification supplied to the Tenderers in the tender stage.

loads up to the limits of serviceability (SLS1 and SLS2).

- Hence the arrow below SLS is indicating that the SLS criteria are defining the state beyond which the character of the allowable damages changes to a new level.
- The following columns are to be interpreted in the same way, with arrows indicating the transition points of the allowed damage.

Table 6: Structural decomposition and allowable damage levels

Macro-level	Meso-level		SLS	ULS	SILS	
	Structures	**Sub-structures**				
Main structural system	Restraint/support system	Foundations of the towers	ND	MD	RD	SD
		Anchor Blocks	ND	MD	RD	SD
		Towers	ND	MD	RD	SD
	Main Suspension system	Main cables	ND	MD	RD	SD
		Saddles	ND	MD	RD	SD
	Secondary Suspension system	Hanger System	ND	MD	RD	SD
		Tie down hangers	ND	MD	RD	SD
		Hanger Groups	DD	RD	SD	SD
	Standard deck	Cross girder	ND	MD	SD	SD
		Rail box girder	ND	MD	SD	SD
		Road box girder	ND	MD	SD	SD
	Special deck zones[7]	End Structures and expansion joints	DD	MD	SD	SD
		Near towers and restraint systems	DD	MD	SD	SD

The limit states and the damage levels to be allowed for are quantified by means of the allowable stress limits indicated in Table 7. These limits are specifically identified for the performance of the following structural components of the main structural system:

[7] They refer to the following components: restraint devices, bearings, expansion joints, that have shorter service life in relation to the Bridge design life, and that can be replaced whilst maintaining the Bridge functionality. The General Contractor shall indicate the service life of such devices.

- main cables,
- hangers.

Table 7: Allowable stresses for components of the suspension system.

Max allowable stress	(SLS2)	(ULS)
Main cables	Ultimate stress/2.10	Ultimate stress /1.67
Hangers	Ultimate stress /1.67	Ultimate stress /1.40

The safety checks at SLS are carried out according to the indications of Par 6.8.
With regard to the strengths and allowable stress levels for the other components of the structural system, reference must be made to the current Italian Standards, refer section 12 References.
Where the Italian Standards do not fully cover the required checks and issues, the General Contractor shall adopt the Eurocodes or other design codes acknowledged at international level. Stretto di Messina S.p.A. retains the right to accept the use of such codes and proposals.

The structural configuration of the Bridge must prevent the progressive propagation of failure mechanisms, by means of a suitable definition, both at the local and global levels, of structural details and the provision of appropriate lines of defence. A suitable structural compartmentation must therefore be sought, if necessary by means of an appropriate arrangement of connections. In particular, the local collapse of a section of the deck structure due to failure of hangers and/or cross beams shall not result in a progressive collapse of the deck.

The Bridge, in its primary and secondary components, must be designed for any aeroelastic phenomenon that may affect it during its design life, with adequate margin with respect to both safety and serviceability aspects. In particular, the critical flutter velocity at deck level (70m asl) must not be less than 75m/s.

4.2 Performance relating to durability

For the purposes of design for durability, materials, structural components and the structure in its entirety must be designed, checked and monitored with reference to phenomena of degradation of their physical and mechanical properties, in particular due to corrosion and fatigue, according to their design service life.
The materials and components of the support and restraint system, the main suspension system

and the decks must have design service life longer than 200 years (design life of the structure).
For structural elements that will be subject to significant degradation of their mechanical properties during the design life of the structure, repairability and replaceability plans must be prepared.
The materials and the components of special deck zones, i.e. lateral and longitudinal restraint devices, movement joints, bearings may have service lives shorter than the Design Life of the Bridge: in such cases their design shall include the definition of suitable procedures for maintenance and replacement, without impairing the performance of the structure.
The same dispositions shall apply for the non structural components of the Bridge, hard shoulders, parapets and safety fences, road pavement, railway tracks, railway gantries and electrical lines: for each a programmed maintenance plan must be prepared, that addresses periodical replacements of components without incurring substantial reductions to the Bridge performance.

4.2.1 Corrosion

All steel surfaces must be adequately protected by adequate paint systems against environmental aggression. The inside surfaces of the deck boxes shall be further protected by means of suitable air dehumidification systems; moreover the programmed maintenance cycles to be carried out must be clearly specified. The wires of the main cables and hangers shall be protected by means of an adequate zinc coating, or otherwise be made of corrosion resistant steel; the cables and the hangers shall be further protected by adequate coating systems. For the main cables, dehumidification systems for the wires shall be provided.

4.2.2 Fatigue

The design life of 200 years of the structure requires, as regards fatigue, a reference to unlimited life for structural components subject to direct man-generated loading: i.e. the rail and road box girders and the cross girders. Over the 200 year design life in fact, these can be subjected to more than 200 million stress cycles. Therefore these components, in order to mitigate the effects of fatigue, will have to be detailed and designed in such a way as to minimise any process of fatigue failure and to readily allow for inspection and checking of welds, joints and connections; the allowable working stress levels must be defined as a function of the fatigue resistance of materials, structural components and connections, according to what prescribed by EN 1993-1-9 [E25], based on laboratory tests with number of cycles of 10 million.

Conventional solutions may be adopted for construction details for which approved laboratory results exist that prove their capacity to withstand the design number of cycles.

The stress ranges to be considered for the safety checks shall be taken from the limiting fatigue curve, referred to the characteristic value σ_k:

$$\sigma_k = \sigma_m - 1.33\Delta,$$

where σ_m and Δ are the mean and the standard deviation values respectively.

The statistical number of tests for the same structural component or connection shall not be less than 12.

Should the welded connections, be unable to guarantee with adequate margin the fatigue resistance for the design life of 200 years, using proven quality assurance procedures, the adoption of bolted connections shall be evaluated. These shall be adequately tested with procedures similar to those of the welded connections.

The structural components shall be verified as follows:

1) The rail box girder shall be designed:
   - 1a) for the loading induced by the heaviest train as indicated in Par. 5.2.1;
   - 1b) for the fatigue degradation induced by the loading combination between the heaviest train as specified by Instruction RFI n. 44F or Eurocode 1 (identified in the train EN5) and one heavy road vehicle LM2 as specified by NTC08.
2) The road box girder shall be designed:
   - 2a) for loadings induced by users as defined in the following Par. 5.2.1;
   - 2b) for the fatigue degradation induced by the loading combination between the heaviest train as specified by the Instruction RFI n. 44F or Eurocode 1 (identified in the train EN5) and one heavy road vehicle LM2 as specified by NTC08.
3) The deck cross girders shall be designed:
   - 3a) loading induced by the railways users as defined in the previous bullet 1a) and by the roadways users as defined in the previous bullet 2a);
   - 3b) for the fatigue decay induced by the loading combination between the heaviest train as specified by the Instruction RFI n. 44F or Eurocode 1 (identified in the train EN5) and one heavy road vehicle LM2 as specified by NTC08.

All other elements that may be affected by significant effects due to fatigue shall be verified according the Italian standards or, where necessary, according other codes recognized internationally.

4.3 Performance relating to functionality (serviceability)

Table 8 shows the general functionality levels identified for the Bridge.
With the overall structure are associated performance levels corresponding to SLS (SLS1 and SLS2), ULS and SILS, as indicated in Table 9.

Table 8: Definition of functionality levels

Functionality Level		Abbreviation	Description
1	Complete functionality	**CF**	Road and railway runability guaranteed
2	Railway functionality	**FF**	Only railway runability guaranteed
3	Lack of functionality	**AF**	Neither road nor railway runability guaranteed

Table 9: Limit states and allowable functionality levels

SLS1	SLS2	ULS	SILS
CF	FF	AF	AF

The state of functionality can be identified when the requirements which define the nominal capacity of the Bridge are satisfied with respect to:

1. railway usage;
2. road usage;
3. marine traffic.

This nominal capacity assumes the form of a set of performance levels for each of the groups of requirements listed above and summarised in Paragraphs 4.3.1, 4.3.2, 4.3.3 that must be satisfied under the most unfavourable combinations (indicated in Chapter 6) of the actions defined in Chapter 5.
The General Contractor shall determine the range of movement joints and bearings, in order to achieve the best overall balance between:

- Variations in the geometry of the structure and dynamic effects at the SLS;

- Restraint forces at the ULS;
- Economy of construction and operation of the devices.

The longitudinal and transverse horizontal movement of the deck, at the abutments and in correspondence of the towers, will have to be resisted by suitable damping/restraining devices.
The railway and roadway expansion joints and bearings shall be designed for the movement range produced by:

a. The free movement due to the most onerous temperature change of the Bridge adequately factored, refer section 5.3.3.
b. The deformation of the dampers controlling the longitudinal deck movement due to the environmental and man-generated actions.

The effect of the cyclical longitudinal movement of the deck shall be taken into account, together with any dynamic component, in the design of the end hinges of the vertical hanger cables.
Similar criteria and rules apply to the dimensioning of the free transverse movement range of the deck at the towers and the associated load transmission/damping devices.

4.3.1 Performance relating to railway usage (runability)

At SLS, all aspects relating to the safety and comfort of trains in circulation, including braking, must be verified.
At ULS, all aspects relating to derailment, overturning and braking of the travelling train must be considered in the structural safety verification.
For the allowable limit values for the safety during running of the train, reference shall be made to the Fiche UIC 518 [R03] and Fiche UIC 518-1 [R04], the ORE studies cited in the appendix to the Fiche UIC 518 [R03]: ORE B55/RP8 [R05], ORE C116/RP3 [R06], ORE C138/RP1 [R07], ORE C138/RP9 [R011], ORE C138/DT66 [R08].
The performance levels for railway usage are summarised in Tables 10 and 11.
In particular, the equivalent longitudinal slope is conventionally defined by the average slope, as follows:

- With reference to an origin of the train centre lines at one end of the Bridge, let x_1 and x_2 indicate the positions of the two ends of the railway load comprising two adjacent trains, such that:

$$[\ \max(x_2 - x_1) = 750m\ ].$$

- For the most unfavourable combination of the two adjacent trains and of the (2+2) road live load loaded lengths, let *v(x)* be the vertical coordinate of a generic point along the centre line of the deck and *v' (x)* be the corresponding value of the tangent in the same point.
- The equivalent gradient in percentage terms is defined in the expression:

$$p = \frac{100}{(x_2 - x_1)} \cdot \int_{x_1}^{x_2} v'(x)dx$$

Table 10: Railway runability performance levels

Performance Levels for Railway Runability		**CF (SLS1)**	**FF (SLS2)**
Design Line speed		120 km/h	Subject to limitations
Maximum service speed in relation to the direct action of wind on the convoys[8]		See Table 10a	
Performance levels for runability and safety of railway traffic	Equivalent longitudinal slope	< 1.80% (one train on one track) < 2.00% (two trains on two different tracks)	< 2.20% (two trains on two different tracks)
	Transverse slope	< 8%	< 10%
	Total rate of change of cant of the track (Short base, bases from 1.3 to 4.5 m)	< 0.250% (0.065% due track tolerances+0.185% due to static and dynamic actions on the structure)	< 0.400%
	Total rate of change of cant (Short base, bases from 4.5 to 20 m)	< 0.200% (0.030% due to track tolerances+0.170% due to static and dynamic actions on the structure)	< 0.275%
	Non-compensated acceleration	< 0.6 m/s^2	< 0.84 m/s^2
	Roll speed	< 0.033 rad/s	< 0.036 rad/s
	Vertical acceleration of the track bed	< 0.7 m/s^2	< 1.00 m/s^2
	Longitudinal acceleration	< 2.5 m/s^2	< 2.5 m/s^2
	Derailment check	Y/P<0.8	Y/P<0.8
	Overturning check	ΔP/P<0.9	ΔP/P<0.9

[8] Maximum service line speed of the railway will be subject to limitations in relation to the direct effects of wind on the railway convoys, following the approach CWC (Characteristic Wind Curve) in accordance to the European Technical Specifications for Interoperability (European Standard EN 14067-6, Technical Specifications for Interoperability (TSIs) High Speed Rolling Stock, Annex G) and considering the favourable conditions permitted on the bridge from the wind barriers.

Table 10a Limitations of the rail service in relation to the direct effects of wind on the convoys

Average wind speed	*Wind gust speed*	*Limitations and restrictions*
0 m/s ≤ V ≤ 30m/s	*0 m/s ≤ V ≤ 42m/s*	*No limitations, maximum speed 120 km/h*
30 m/s < V ≤ 38m/s	*42 m/s < V ≤ 54m/s*	*Maximum train speed 60km/h*
38 m/s < V ≤ 47m/s	*54 m/s < V ≤ 67m/s*	*Progressive closure depending on the train type*
47 m/s < V	*67 m/s < V*	*Complete traffic closure*

Table 11: Railway comfort performance levels

Performance Levels for comfort		**CF (SLS1)**	**FF (SLS2)**
Comfort and vehicle/structure interaction performance levels	Comfort index Wz	< 2.2	-
	Peak vehicle acceleration b_v	≤ 2 m/s^2	-
	Rms transverse acceleration (passenger trains)	< 0.5 m/s^2	-
	Rms vertical acceleration (passenger trains)	< 0.75 m/s^2	-
	Recoil	0.25 m/s3	0.58 m/s^3

The accelerations of the railway tracks are to be intended in the absence of seismic actions.
The kinematic effects at movement joints shall also be checked, making sure that the calculated movements are compatible with the design solution adopted.
It is recommended that the effects of dynamic interaction induced by the road traffic on the railway be checked and documented, for the purposes of checking runability.
The analyses of dynamic railway runability in presence of earthquake and wind shall verify safe railway runability, in relation to the type of trackbed adopted, and in particular:

a. At SLS1, all aspects relating to safety of railway traffic, including braking;
b. At SLS2, derailment, overturning, reference Gabarit kinematic envelope and braking.

Particular care shall be used in the definition of a suitable guardrail system running alongside the railway tracks, which shall be designed in such a way that the concentrated load due to the impact of the rolling stock can be resisted. Adequate attention shall also be paid to the provision of support points for a lifting system of derailed rolling stock.
The tracks shall be made of UIC60-900A rails, laid at an inclination of 1:20, nominal gauge of 1435 mm and laying tolerance of -1 mm, +3 mm. The trackbed system shall be unballasted but based

upon a continuous elastomeric support placed between the rails and the structure, of the "embedded rail" type or equivalent. The quality of the rail will be defined for a velocity v in the range 90<v<130 Km/h.

4.3.2 Performance relating to road usage

Table 12 below shows the performance levels expected for road usage.
The accelerations of the road surface are to be intended in absence of seismic actions.

Table 12: Roadway runability performance.

Performance Levels for runability		CF
Design speed		90 Km/h
Performance levels for runability and safety of traffic	Total Longitudinal slope	< 5%
	Total Transverse slope	< 7%
	Horizontal acceleration of the roadway (f < 1 Hz)	< 0.5 m/s^2
	Vertical acceleration of the roadway (f < 1 Hz)	< 1.5 m/s^2

The detailing and design must provide for the appropriate drainage of rain water.
The kinematic effects at the end joints shall also be checked, making sure that the calculated displacements are compatible with the design solution adopted.
It is recommended that the effects of dynamic interaction induced by the rail traffic on the roadway be checked, for the purposes of guaranteeing runability.
Up to a wind reference speed of 44m/s at deck level (70m asl) (SLS1) the accelerations of the deck shall be compatible with the values indicated in Table 12.
The wind shields with aerodynamic profiles to be provided along the edges of the deck shall fulfil the following requirement: The total pressure loss coefficient, K_s must be equal to 2.7 within a 5 % margin.

4.3.3 Performance relating to marine traffic

The functionality in use shall guarantee that the navigation clearance is respected. This performance is associated with the complete functionality of the structure (CF).
The term “Basic design profile” (Linea Fondamentale di Progetto) refers to the vertical profile of the

road alignment.

The term navigation clearance, refers to the minimum vertical distance between the deck soffit and the mean sea level, as geometrically defined in the Progetto Preliminare, which is to be 65m over a central width of 600m and 50m in the outer remaining parts of the water channel.

The navigation clearance shall be satisfied by the deflected deck configuration, starting from the Basic Design Profile, at the reference temperature (refer to section 5.3.3), and under highway loading QR (as per Par.5.2.2) and railway loading comprising:

a. 2 real trains RFI 5 as from Par. 5.2.2 having a length of 400m each;

b. 1 real train RFI 5, as the above, having a length of 750m.

The deflected shape of the Bridge, in relation to the roadway, railway and marine traffic needs, shall be monitored, controlled and managed.

5. Definition of actions

The design and performance checks of the Bridge shall consider at least the actions indicated in Table 13.

The uncoupling of spatial distribution and time variability of actions, intrinsically described by functions of space and time, is a simplification that is permissible only if adequately justified and accepted by Stretto di Messina S.p.A. Possible correlations between the space-time functions which describe two different actions must be justified in the design phase and are subject to the approval of Stretto di Messina S.p.A.

Table 13: Design Actions

<table>
<tr><td rowspan="3">Par. 5.1</td><td colspan="3">Permanent Actions (P)</td></tr>
<tr><td colspan="2">Structural Self Weight</td><td>PP</td></tr>
<tr><td colspan="2">Self Weight of non-structural elements</td><td>PN</td></tr>
<tr><td rowspan="4">Par. 5.2</td><td colspan="3">Variable man-generated actions (Q)</td></tr>
<tr><td colspan="2">Actions for local sizing of the structural system (strength and deformation at the micro- and meso- level[9])</td><td>QL</td></tr>
<tr><td rowspan="2">Actions for the global sizing of the structural system and for serviceability checks (strength and deformation at the macro-level[10])</td><td>Dense variable load</td><td>QA</td></tr>
<tr><td>Rarefied variable load</td><td>QR</td></tr>
<tr><td rowspan="5">Par. 5.3</td><td colspan="3">Variable natural and environmental actions (V)</td></tr>
<tr><td colspan="2">Wind action</td><td>VV</td></tr>
<tr><td colspan="2">Seismic action</td><td>VS</td></tr>
<tr><td colspan="2">Thermal action</td><td>VT</td></tr>
<tr><td colspan="2">Water in drainage pipes (rain)</td><td>VR</td></tr>
<tr><td>Par. 5.4</td><td colspan="2">Accidental actions (A)</td><td>A</td></tr>
</table>

Considering the scale of the structure, in general terms different intensity levels shall be recognised for the variable actions.

In particular, for man-generated actions, two sets of loadings are identified for two structural scales:

- Loadings for the design and performance checks of the main structural system (macrolevel): suspension system (main cables and saddles), support/restraint system (towers, foundations, anchorages) (Par. 5.2.2)
- Loadings for the design and performance checks at lower levels (meso and micro levels) (Par. 5.2.1)

At the lower levels, characterised by geometric dimensions less than 300 metres, greater load

[9] QL load is applied for local design of road and rail box girders, cross girders, hangers and hanger groups. Local verifications shall include the effects of global wind actions (VV) and thermal action (VT) from global analysis.

[10] QA and QR load is applied for global design of the restraint and support system, main cables, hanger system, saddles and tie down hangers, as well as the deck in those cases where global loading should result as governing for the design.

intensities shall be considered, and shall be considered in accordance with current Italian Standards (refer section 12 References), including as regards their mode of application.
The same approach of increasing the local loading intensity shall be applied to the wind load, for which appropriate local amplification factors shall be evaluated in terms of gust factors.
Should the General Contractor discover any omission, lack of clarity, inconsistency or ambiguity in the definition of the loading actions and their combinations given in this document, they shall refer this to Stretto di Messina S.p.A. in order to resolve any such equivocal issue with the explicit consent of Stretto di Messina S.p.A.

5.1 Permanent actions (PP and PN)

The geometric configuration defined in the design of the Bridge structure clearly defines the spatial distribution and intensity of the dead loads (self weight), which do not vary at the different checking levels envisaged.
Permanent and semi permanent actions are broken down into structural self weight (PP) and non-structural self weight (PN), as detailed below:

- **PP**: the structural weight must be calculated for all structural components (including fill in anchor blocks), including connections, coatings, and all structural provisions for Bridge services and maintenance.
- **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, shall be considered.
- In addition to the self weight loads of above mentioned components, which shall be calculated analytically, an additional load of 1.5kN/m uniformly distributed along the axis of each deck box shall be taken into account as a PN load.

5.2 Variable man-generated actions

The variable man-generated actions to be considered in the sizing the Bridge are broken down as follows:

1. Actions for local sizing of the structural system (strength and deformation at the micro- and meso- levels)
2. Actions for the global sizing of the structural system and for serviceability checks (strength and deformation at the macro-level)

The highway and railway loads shall be disposed along the Bridge in such a way as to produce the most adverse load effect, according to the different performance effects to be checked and the different levels of structural decomposition considered.

5.2.1 Actions for local sizing of the structural system (strength and deformability at micro- and meso- levels) (QL).

The main deck structure (road and rail box girders, cross girders) and the secondary elements of the suspension system (hangers and hanger groups) shall be designed, as far as the loading is concerned, in accordance with the following:

- Roadway: in accordance with current Italian Standards, in particular D.M. 14.1.2008 (NTC 08) [S18]
- Railway: in accordance to RFI DTC-ICI-PO SP INF 001 A, dated 12/10/09 [R10] ("Superimposed loads for the design of railway bridges – Instructions for design, construction and load testing").

The structural elements of the lateral service lanes, placed outside the external edges of the deck, shall be checked under the following load conditions:

a. Live load equal to 5 kN/m2 - uniformly distributed;

b. Service equipment/crane, with two axles respectively loaded by 80 kN and 40 kN, 1.30 m wide, 3.00 m spaced, and load pads of 0.20m x 0.20m.

Such local load is defined in order to design the secondary elements only, and it is not to be taken into account in any load combination for global checks.

5.2.2 Actions for global sizing of the structural system (strength and deformability at the macro-level) (QA and QR)

With reference to the global system, a distinction is made between:

- actions to be considered for the purposes of assessing the load-bearing capacity of the restraint and support system, main cables, saddles, hanger system, tie down hangers and of the deck for those cases in which global loading should result governing for the design (QA);
- actions to be considered in the assessment of structural response in terms of deformability as it affects runnability (QR).

To this end, two sets of traffic loading are introduced, to be disposed along the Bridge in the most unfavourable loading pattern for the load effects being checked and the structural level of decomposition considered:

1. Dense variable load (QA);

The following are considered:

- for each of the two carriageways,
  - most heavily loaded lane: a distributed line load of intensity 15 kN/m,
  - each of the other lanes: a distributed line load of intensity 5 kN/m,

  These shall be arranged along the centre lines of the lanes in the most unfavourable pattern, possibly as a combination of adverse areas. As far as the braking load is concerned, it will be assumed to act in the direction of the road centreline, at the level of the road surface , and with intensity equal to 1/10 of the associated uniformly distributed line load.

- for each of the two railway tracks, two loaded trains, each 750m long and having a total intensity of $\alpha \cdot q$ = 1.1 x 80 = 88 kN/m; the distance between the two trains will be not less than 750m.

  The characteristic braking and acceleration loads, to be combined with the vertical loads, shall be as follows, where L is length of loaded track that gives the worst effect:

  - acceleration: 33(kN/m) x L(m) up to a maximum of 1000 kN
  - braking: 20(kN/m) x L(m) up to a maximum of 6000 kN

  A scenario with two trains travelling in two opposite directions shall also be considered, including the case of one train braking and the other accelerating.

2. Rarefied variable load (QR);

The following loads shall be considered:

- for each of the two carriageways,
  - most heavily loaded lane: a distributed line load of intensity 3.75 kN/m,
  - one of the other two lanes: a distributed line load of intensity 1.25 kN/m,

  These shall be disposed along the centre lines of the lanes in the most unfavourable pattern, possibly as a combination of adverse areas. As far as the braking load is concerned, it shall be assumed to act in the direction of the road centreline, at the level of the road surface, and with an intensity equal to 1/10 of the associated uniformly distributed line load.

- for each of the two railways, one loaded train, 750 m long and total intensity of $\alpha \cdot q$ = 1.1 x 80 = 88 kN/m. The characteristic braking and acceleration loads, to be combined with the vertical loads, shall be as follows, where L is length of loaded track that gives the worst effect:
  - acceleration: 33(kN/m)*L(m) up to a maximum of 1000kN
  - braking: 20(kN/m)*L(m) up to a maximum o 6000kN

  A scenario with two trains travelling in two opposite directions shall also be considered, including the case with one train braking and the other accelerating.

5.2.3 Real trains

Specific runability analyses in dynamic conditions shall be carried out using 6 train types representing the actual trains circulating on the RFI rail network:

- Train RF1 (type AV with 18t/axle)
- Train RF2 (type AV Tilting with 14t/axle)
- Train RF3 (IC with locomotive 25t/axle and carriages 18t/axle)
- Train RF4 (high frequency double decker train with 20t/axle)
- Train RF5 (goods train with locomotive 25t/axis and carriages 25t/axle)
- Train RF6 (goods train with unloaded carriages with 1.25t/axle and locomotive 25t/axle)

The characteristics of the above mentioned 6 trains are defined in Annex 2 to this document.
In particular, the analyses shall be conducted using the 6 actual RFI trains, taking into account the dynamic characteristics of the train such as masses and their relative distribution in space, construction characteristics of the vehicles, and elastic and damping characteristics of the suspensions. These analyses shall also enable checking the compliance with the expected comfort levels indicated in Table 10 and 11.

5.3 Variable environmental actions due to natural phenomena

These actions are characterised by variability in the space-time field. The intensity and associated direction of the individual actions depend on the return period in question.
For each of the loading levels referred to in Paragraph 3.2 the respective intensities are defined, each of which is variable along the longitudinal axis of the Bridge or along the vertical axis of the towers.
The variability with time is considered through the definition of time histories.

5.3.1 Wind action (VV)

The space-time characterisation of the wind used for the design of the structure is hereafter illustrated.
Where the mean wind speed over an interval of 10 minutes at height z is defined as $\bar{u}$, the vertical trend is given in the following formula:

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

The symbols contained in the above formula have the meaning indicated in Table 14:

Table 14: Definition of the parameters which describe the vertical trend in mean wind speed.

Symbol	Description	Value	
z	Height (in metres) above sea or ground level	$z \geq 2$ m	
α_d	Wind direction factor	$\alpha_d=1$	
α_r	Return factor, associated with the checking level	SLS1	α_r =1.00
		SLS2	α_r =1.07
		ULS	α_r =1.21
		SILS	α_r =1.35
z_o	Standard roughness length[11]	z_o =0.01 m	
k_r	Roughness coefficient	k_r = 0.17	
u_{ref}	Reference mean wind speed (Maximum value for a return period of 50 years, 10 m from the ground, on flat even ground with roughness length $z_{o\,ref}$= 0.05 m)	29 m/s	

Hence, at the level of 70 m above sea level the 4 levels of velocity of Table 15 are determined.

Table 15: Levels of velocity and return period associated

SLS1	SLS2	ULS	SILS
44 m/s	47 m/s	54 m/s	60 m/s

Given the exceptional length of the Bridge, it is recommended that the change in the mean wind speed along the longitudinal axis of the deck be assessed and considered in a suitable and justified manner.

Atmospheric turbulence may be broken down into three components hereinafter called longitudinal turbulence (u, horizontal, in the x direction, parallel to the mean wind speed), lateral turbulence (v, horizontal, in the y direction, perpendicular to the mean wind speed) and vertical turbulence (w, in the vertical direction z). These are defined by means of the corresponding power spectra, intensity, integral scales and coherences described in the Annex 1 to this document.
The provided wind description corresponds to intense values of mean speed, under which the atmosphere tends to stratify into a neutral regime. For frequent, and therefore moderate, mean

[11] With regard to the assessment of the turbulence a conventional roughness length Zo = 0.05m shall be assumed.

wind speed values, thermal stratification varies from case to case and thus causes changes (some significant) to the mean wind speed and turbulence profiles. It is recommended that account be taken of possible turbulence amplifications for instable thermal conditions or turbulence absence for stable thermal conditions, in a suitable and justified manner.
Mean wind speed and atmospheric turbulence produce, on the primary and secondary structural components of the Bridge, a system of aerodynamic and aero elastic actions whose definition requires the assessment of aerodynamic coefficients of drag, lift and moment, of aerodynamic admittance functions and of aero elastic derivatives. These values may be determined by means of specific wind tunnel tests carried out at highly qualified laboratories. It is recommended that these tests be conducted and certified by at least two independent laboratories, in addition to those used by Stretto di Messina S.p.A. The use of assessments made using CFD (Computational Fluid Dynamics) numerical codes, which is highly recommended, may represent a useful support and complement to wind tunnel tests. On no account may the latter be replaced. Suitable numerical simulations must be performed, in which account is taken of the space-time non-correlation of the action induced by wind. For the SLS, the ULS and the SILS the performance behaviour of the structure must be assessed also using the space-time histories developed by Stretto di Messina S.p.A., attached to the document DT.ISP.V.E.R1.001 “Valutazione del vento di progetto” (“Definition of the design wind”) included in the “Studi e documenti informativi” (“Studies and informative documents”), with following integrations and amendments.

In a similar way as for the variable man-generated actions, the definition of wind actions for sizing the micro-level structural components involves determining equivalent static actions, the expression of which contains the wind speed multiplied by an appropriate gust factor, to be determined on a case by case basis and to explicitly indicate and justify. This factor on no account includes the contribution of any local resonance effects, which must be considered in parallel studies. The equivalent static actions derived will be subject to checking in the light of the results of the non-linear global dynamic analysis.

5.3.2 Seismic action (VS)

The design seismic motion is defined through the response spectra of the horizontal and vertical components, in accordance with Tables 16 and 17. Period T is expressed in seconds, whilst the ordinate gives the pseudovelocity.

Table 16: *Horizontal component of the seismic response spectrum.*

Damping	$\rho = 0.05$
	Pseudo Spectrum Velocity - Horizontal
$T < 0.03\ sec$	$PSV = a_p \cdot \frac{T}{2\pi} \cdot 1.0$
$0.03\ sec < T < 0.10\ sec = T_1$	$PSV = a_p \cdot \frac{T}{2\pi} \cdot \frac{(T_2 - T) + 2.5 \cdot (T - T_1)}{T_2 - T_1}$
$0.10\ sec < T < 0.65\ sec = T_2$	$PSV = a_p \cdot \frac{T}{2\pi} \cdot 2.5$
$0.65\ sec < T < 3.50\ sec = T_3$	$PSV = a_p \cdot 1.0 \cdot \frac{T_2}{2\pi} \cdot 2.5 \quad (= 0.25863 \cdot a_p)$
$3.50\ sec < T < 10.0\ sec = T_4$	$PSV = a_p \cdot \frac{2\pi}{T} \cdot \frac{T_2}{2\pi} \cdot \frac{T_3}{2\pi} \cdot 2.5 \quad \left(= 0.14407 \cdot a_p \cdot \frac{2\pi}{T}\right)$
$10.0\ sec < T < 30.0\ sec = T_5$	$PSV = a_p \cdot \frac{2\pi}{T} \cdot \frac{T_2}{2\pi} \cdot \frac{T_3}{2\pi} \cdot 2.5 \cdot \frac{0.62120 \cdot (T_5 - T) + 0.60102 \cdot (T - T_4)}{0.62120 \cdot (T_5 - T_4)}$
$30.0\ sec < T$	$PSV = a_p \cdot \frac{2\pi}{T} \cdot \frac{T_2}{2\pi} \cdot \frac{T_3}{2\pi} \cdot 2.5 \cdot \frac{0.60102}{0.62120} \quad \left(= 0.10545 \cdot a_p \cdot \frac{2\pi}{T}\right)$

Table 17: *Vertical component of the seismic response spectrum.*

Damping	$\rho = 0.05$
	Pseudo Spectrum Velocity - Vertical
$T < 0.03\ sec$	$PSV = a_p \cdot \frac{T}{2\pi} \cdot 1.0$
$0.03\ sec < T < 0.06\ sec = T_1$	$PSV = a_p \cdot \frac{T}{2\pi} \cdot \frac{(T_2 - T) + 2.5 \cdot (T - T_1)}{T_2 - T_1}$
$0.06\ sec < T < 0.40\ sec = T_2$	$PSV = a_p \cdot \frac{T}{2\pi} \cdot 2.5$
$0.40\ sec < T < 5.00\ sec = T_3$	$PSV = a_p \cdot 1.0 \cdot \frac{T_2}{2\pi} \cdot 2.5 \quad (= 0.15916 \cdot a_p)$
$5.00\ sec < T$	$PSV = a_p \cdot \frac{2\pi}{T} \cdot \frac{T_2}{2\pi} \cdot \frac{T_3}{2\pi} \cdot 2.5 \quad \left(= 0.12665 \cdot a_p \cdot \frac{2\pi}{T}\right)$

The four peak ground acceleration levels a_p (PGA), associated with each checking level, are shown in Table 18:

Table 18: Peak ground acceleration levels.

SLS1	SLS2	ULS	SILS
1.2 m/s^2	2.6 m/s^2	5.7 m/s^2	6.3 m/s^2

Suitable numerical simulations shall be performed, in which account is taken of the space-time non-correlation between the seismic actions transmitted to the foundations of the towers and to the anchor blocks of the suspension system.

For the Ultimate Limit States (ULS) and Structural Integrity Limit States (SILS), the Bridge shall be checked by using space-time ground acceleration histories that are compatible and coherent with the spectra defined above. The overall collection of time histories shall contain those developed by Stretto di Messina S.p.A and attached to the document DT.ISP.S.I.R2.001 “Storie temporali dell’azione sismica” (“Time histories of seismic actions”), included in the “Studi e documenti informativi” (“Studies and infomative documents”).

5.3.3 Thermal action (VT)

The computation of the thermal loading and of the deformations induced on the structure must be based on the diagram of the conventional temperature time history of the air. This shall be referred to at least seven consecutive daily cycles. The calculation shall be carried out at least for both the condition of maximum summer air temperature and minimum winter air temperature. Effective temperatures, deformations and stresses of the structural elements shall be derived from the air thermal state, taking into account the variation of air temperature with height, solar radiation, air velocity and the other mechanisms of thermal exchange.

In absence of more accurate and proved information, subject to the approval of Stretto di Messina S.p.A., the thermal state corresponding to the daily cycles described in Table 19 shall be adopted for the prescribed levels of structural verification.

Particular attention shall be paid to the effects caused by the mass of air contained inside the box girders.

Table 19: Reference daily air temperature at 10m above seal level (°C).

Season	Level	Hour in the day											
		0	**2**	**4**	**6**	**8**	**10**	**12**	**14**	**16**	**18**	**20**	**22**
Summer	SLS	27.0	25.0	23.0	22.5	27.0	33.0	38.5	41.0	38.5	33.5	31.0	29.0
	ULS	28.5	26.5	24.5	24.0	28.0	34.5	40.0	42.5	40.0	35.0	32.5	31.0
	SILS	29.0	26.5	24.5	24.0	28.5	36.0	41.5	44.0	42.0	36.0	33.5	32.0
Winter	SLS	2.0	1.5	0.5	-0.5	0.0	5.5	12.0	12.0	6.0	5.0	4.0	3.0
	ULS	1.0	0.0	-1.0	-1.5	-1.5	5.0	11.0	11.5	5.0	3.5	3.0	2.0
	SILS	0.0	-0.5	-2.0	-2.5	-2.0	4.0	10.0	10.5	5.0	3.0	2.0	1.0

5.4 Accidental actions (A)

Accidental actions are understood to mean those actions whose presence cannot be ruled out altogether, and whose likelihood cannot be interpreted in a statistical context.

The design of the structure prepared by the General Contractor shall contain the results of in depth structural analyses referring to accidental loading conditions that may affect the structure and lead to its complete or partial collapse. In checking these, the accidental load conditions shall be considered as not acting simultaneously.

The design shall include:

- a complete identification of the possible accidental load conditions, in addition to those indicated below;
- the damage levels and the arrangements to be adopted in each case;
- clear reference to the preventive and emergency measures, intended to limit the instantaneous and progressive effects of local failure, up to the global collapse;

In particular, the General Contractor shall at least consider the effects of the following phenomena, according to different structural loadings, in conformity with what indicated in Table 20: the pertinence of the checks at the ULS and SILS shall be determined coherently with the importance of the accidental action considered.

Table 20: Accidental Live Loads and limit states

Accidental load	Limit State
Fire	ULS / SILS
Explosion	
Impact	

5.4.1 Safety in the event of fires and explosions

5.4.1.1 Fire

The General Contractor shall submit a study of the effects of accidental fire by means of a procedure articulated as follows:

- Definition of the significant design fire scenarios
- Definition of the design fire events corresponding to the scenarios defined above
- Definition of the space-time distribution of temperature in the structural components
- Evaluation of the evolution in time of the damage and the structural behaviour both at a global and a local level, by means of incremental non-linear analyses.

5.4.1.2 Explosion

The General Contractor shall submit a study of the effects of accidental explosions on the structure, with particular reference to the space-time distribution of pressures on the structure and to the mechanisms of diffusion of the stress and deformation states, as well as that of the damage up to the collapse, be it local or global. The analyses shall be conducted in dynamic conditions, based on documented assumptions on the modality and intensity of the impulsive loading action. The documents relating to these studies shall have reserved character and limited distribution only after the approval of Stretto di Messina S.p.A.

5.4.2 Impact

The effects of damage due to impact shall be analysed and documented with regard to the following circumstances:

- Ship impact on the foundations and the towers: the loading action to be considered can be defined according to the recommendations of prEN 1991-1-7, taking into account the distance between the protection bank and the tower.
- Airplane impact: the airplane to be considered is related to the robustness of the Bridge structure and the event likelihood; the valuation of the robustness of the main structural components shall be done with reference to a commercial airplane of 10 tonnes and with impact velocity of 600 km/h.

The possible impact actions shall be considered at different locations, with different intensities, and under different structural behaviours. Therefore the possible space-time distributions of the stress and deformation states, as well as that of the damages up to the collapse, be it local or global, shall be determined.

The analyses shall be carried out in dynamic conditions, based on realistic and documented assumptions on the mass, the shape, the velocity and the area of contact between the structure and the impacting airplane.

The most sensitive sections of the structural system shall be determined, as well as the critical energy levels in relation to the preservation of the state of structural integrity, up to the onset of the mechanism of global collapse, according to the different schematic hypotheses of impact determined.

6. Analysis criteria, load and contingency scenarios

The configuration and dimensions of the Bridge determine a variability in the geometry assumed by the structure due to the structural stress, induced by variable environmental/natural and man-generated actions, that is governing the static and dynamic response of the structure. Many structural components, being particularly slender, are consequently prone to instability behaviour, while others, such as restraint devices and movement joints, are characterised by non-linear mechanical behaviour.

These observations make evident the intrinsically non-linear behaviour of the structure, which the analysis must take into account. As a consequence the verification of performance levels of the Bridge shall be carried out by means of non-linear analyses. Analyses done in the linearised domain may be used to determine the most restrictive loading scenarios.

All the assumptions and steps in the theoretical formulation and numerical implementation of the modelling shall be clearly documented. Sections or components potentially preferred for the assessment of the performance shall also be specified; these choices will also have to be justified.

6.1 Superposition of load effects

The intrinsic non-linear behaviour of the structure and the inadequacy of models based on linear or linearised approximations imply that:

it is not possible to evaluate the effects of a load combination by superimposing the effects of each individual load, but it will be necessary to run analyses for every likely sequence of loads to be combined; the loading actions shall be applied in the most unfavourable sequence and be increased progressively, up to their ultimate value; the ULS strength checks shall be carried out in order to determine the limit amplification factor, by progressively increasing the load multiplier.

6.2 Sensitivity to imperfections

The intrinsic non-linear behaviour and the presence of slender compressed members make the structure potentially subject to buckling phenomena and sensitive to the presence of geometrical imperfections (joint misalignment, defects of planarity, etc.) and to the departure of the load distribution from the ideal configuration assumed in the design.

The analyses shall highlight the degree of sensitivity of the structure to structural imperfections, in its overall behaviour and in its individual components, at the various structural levels.

As a consequence of the above considerations, the following recommendations hold:

- buckling checks shall be carried out into the post-critical range, allowing for the possible interactions and coupling between the potential different critical modes;
- local checks on single components or parts of the structural system shall take into account the interaction with the rest of the structure;
- a statistic analysis shall be conducted of the different structural elements to be erected in order to evaluate the realistic possible imperfections and therefore define ‘design imperfections’;
- the effects of imperfections shall be investigated by means of a sensitivity analysis with respect to imperfections, possibly using algorithms of the perturbative type;

In particular, structural analyses shall be performed taking into account possible errors in verticality and straightness of the tower legs, in both the longitudinal and transverse planes, to be considered together with the other permanent actions. The deviations from verticality to be considered in the analyses shall not be less than the values in Table 21, in which H identifies the height of the towers above their foundation level.

Table 21: Design curvatures and deviations from verticality of the tower legs.

• Max longitudinal deviation of the tower top	H/1000
• Max transverse deviation of the tower top	H/1000
• Max longitudinal deviation at the tower mid-height from the theoretical vertical line	H/2000
• Max transverse deviation at the tower mid-height from the theoretical vertical line	H/2000

It remains understood that the imperfections referred to above are for the sole purpose of ensuring the robustness and safety of the design proposed by the General Contractor. Particular attention shall be paid to the definition and analysis of the residual stress and deformation states produced by the construction process. Irreversible deformations and geometric distortions of the structural components of the Bridge as a result of the construction that leads to a decrease in the performance levels will not be accepted.

Should the values of Table 21 be exceeded, the General Contractor will have to revaluate the loadings, the structural response, safety and performance of the structure in the actual configuration of the structure at the end of construction and shall be deemed responsible for the imperfections present in the structure “as built”. Stretto di Messina S.p.A. retains the right to accept any imperfection that should exceed the values of Table 21.

6.3 Dynamic analysis

The dynamic nature of the environmental and man-generated loading actions, together with the non-linear behaviour of the structure, means that in depth structural analyses must be conducted in the dynamic range, using step by step numerical methods. The time integration algorithms used for each analysis shall be specified. The assessment of safety and serviceability obtained with static analyses will also have to be complemented by analyses in the dynamic non-linear range.

6.4 Use of multilevel analysis

Taking into account the exceptional dimensions and complexity of the Bridge structure, describable within the framework of the three different levels of description indicated in Chapter 2 (Macrolevel, Mesolevel, Microlevel), a global analysis of the structure shall be based on the most refined level of description appropriate. Furthermore, it is necessary to use a hierarchy of numerical analyses, each tailored for the most efficient level of description for the evaluation of the single effects of interest, and therefore to employ a structured management, of multilevel type, of the analyses. In this approach, the results of less refined models are to be considered as a solution of first approximation for the most refined models. The various levels of analysis will have to show consistence of results.

6.5 Robustness checks

The structural robustness of the Bridge must be investigated in detail, i.e. its capacity to undergo only limited reductions in its performance levels in the event of departures from the original design configuration as a result of local damage due to accidental loads or secondary structural elements being out of service for maintenance purposes or degradation of their mechanical properties.

The structural robustness of the Bridge under extreme live and environmental actions is ascertained with the ULS and SILS checks, in which are considered accidental loads of different nominal intensities.

In general terms, the following recommendations apply:

- appropriate contingency scenarios shall be identified, i.e. scenarios of possible damage together with suitable load scenarios, able to characterise Bridge robustness in the various conditions of service;
- analyses shall be conducted, with the aim to characterise the structural safety and performance levels of the Bridge in these conditions.

In order to evaluate the sensitivity of the structure with respect to each single loading action, including those relating to the two types of live load, it is required that load scenarios that comprise a single action also be investigated.

For the definition and the management of the robustness checks, the development of accurate and complete analyses is required, relative to:

- F.M.E.A. - Failure Modes and Effects Analysis
- F.T.A. – Fault Tree Analysis

The results shall be documented and critically analysed by the General Contractor.

6.6 Validation of the design calculation codes

The structural analyses and checking processes and calculations must be clearly and univocally organised and documented, in order to allow a complete traceability of the logical and numerical process.

The different calculation packages used, both main and secondary ones, with their relative versions, will have to be explicitly revealed by the General Contractor.

The general format and the limits of applicability of the calculation codes used will have to be adequately documented. Cross checks will have to be done by means of known solutions or results obtained with independent analyses.

The use of a single calculation code for the analysis of such a structure does not guarantee an adequate reliability of the results. Cross checks with more codes or packages, possibly open to the user so that they an be structured and calibrated for the particular structure analysed, is considered necessary in order to guarantee the “robustness” of the structural analyses.

In general infact, the numerical solution of the structural problem depends on the dimension, shape and orientation of the mesh and the model may lead to non realistic representations of the structural problem (lack of objectivity): by robust formulation it is intended an analysis process that gives ample guarantee of convergence to a mechanically correct solution, if it exists, or that otherwise can show the impossibility to obtain an equilibrium coherent with the structural capacity of the structure.

It is essential to develop analysis and checking processes that allow limiting the extreme values of the structural response to the given loading scenarios and boundary conditions, also for the sensitivity analyses and overall optimisation process of the structure.

6.7 Load and contingency scenarios

In general the following are defined:

- Scenario: an organised and consistent set of situations in which the structure may find itself during its design life.
- Loading scenario: an organised and consistent set of loading actions that may act upon the structure.
- Contingency scenario: a possible limit state for the Bridge determined by a set of loading actions (loading scenario) and applied to a particular structural configuration.

The required performance levels will have to be met by the structure in its nominal configuration, which refers to the structural scheme as defined in every part by the design documents, and in the deformed configurations under variable loads.

The structural analyses shall take into account the effects due to the interaction between the soil and the structure, both for in its vertical and horizontal components, for the foundations of the towers and the cable anchorage structures.

The non-linearity of the behaviour of the structure implies that the most unfavourable load combinations for a single load effect to be verified are not known a priori. Therefore, a significant and reliable set of scenarios will have to be investigated, including those characterised by the absence of one or both of the Bridge live traffic loads.

The following recommendations apply:

- for each limit state considered and for each performance effect, load scenarios must be identified, i.e. sets of loads, organised and consistent in space and time, which represent the realistically possible combinations that are most likely to be critical for the performance effect considered;

- particular attention shall be paid to environmental actions, in particular those due to earthquakes and wind, as a consequence of their variability and complexity;
- the definition, development and validation of the loading scenarios shall be performed by the General Contractor and must be accepted by Stretto di Messina S.p.A.

6.8 Combination of actions

The intensities of the actions to be included in each combination are determined by multiplying the values defined in the previous Chapter 5 by the factors given in Tables 22, 23, 24 and 25 for the Serviceability Limit States (SLS1, SLS2), Ultimate Limit State (ULS) and Structural Integrity Limit State (SILS) respectively.

1. Serviceability Limit States (SLS)

The checks shall be carried out both with respect to serviceability and structural safety aspects. The factors to be applied to the loads for the serviceability checks are given in Table 22; in particular, Par. 4.4.3 refers to the serviceability checks for the case of marine traffic. The factors to be applied to the loads for the structural safety checks are given in Table 23.

In addition to the situations defined in Table 22 and 23, it will be necessary to check the serviceability performance in conditions of programmed maintenance operations, with particular attention to the stress and deformation states induced in the structure during the replacement of hangers as well as bearing and restraint devices.

Table 22: Combinations corresponding to Serviceability Limit States (SLS) for serviceability checks (Road/rail traffic).

Combination	PP	PN	QR	QA	VV	VS	VT	VR	A
1	1.0	1.0	-	-	-	-	0/1.0	-	-
2	1.0	1.0	1.0	-	-	-	0/1.0	-	-
3	1.0	1.0	-	-	1.0	-	0/1.0	-	-
4	1.0	1.0	1.0	-	1.0	-	0/1.0	-	-
5	1.0	1.0	1.0	-	-	1.0	0/1.0	-	-
6	1.0	1.0	1.0	-	-	-	0/1.0	1.0	-

Table 23: Combinations corresponding to Serviceability Limit States (SLS) for stress levels checks.

Combination	PP	PN	QR	QA	VV	VS	VT	VR	A
1	1	0/1	-	-	-	-	0/1	-	-
2	1	0/1	-	-	1	-	0/1	-	-
3	1	0/1	-	-	-	1	0/1	-	-
4	1	1	-	1	-	-	0/1	-	-
5	1	1	-	1	1	-	0/1	-	-
6	1	1	-	1	-	1	0/1	-	-
7	1	1	-	1	-	-	0/1	1.0	-

2. Ultimate Limit States (ULS)

The factors to be applied to the loads for the ULS checks are defined in Table 24. The factor μ must be taken as 1.15 if it refers to the weight of steel elements and 1.25 if it refers to concrete structural elements when the effect of the selfweight is considered to have an adverse effect, while it is to be taken as 0.95, for both materials, if it has a relieving effect.

In addition to the accidental loads defined in Par. 5.4, it will be necessary to consider the contingency scenarios that envisage the failure of the support of one extremity of a cross beam, at the most unfavorable location along the Bridge. The analysis shall be done in the dynamic region, assuming the instantaneous rupture of the support.

Table 24: Combinations corresponding to Ultimate Limit States (ULS).

Combination	PP	PN	QR	QA	VV	VS	VT	VR	A
1	μ	0/1.5	-	-	-	-	0/1	-	-
2	μ	0/1.5	-	-	1	-	0/1	-	-
3	μ	0/1.5	-	-	-	1	0/1	-	-
4	μ	0/1.5	-	-	-	-	0/1	-	1
5	μ	0.9/1.5	-	1.5	-	-	0/1	-	-
6	μ	0.9/1.5	-	1.1	1	-	0/1	-	-
7	μ	0.9/1.5	-	1.1	-	1	0/1	-	-
8	μ	0.9/1.5	1	-	-	-	0/1	-	1
9	μ	0.9/1.5	-	1.1	-	-	0/1	1	-

3. Structural Integrity Limit States (SILS)

The factors to be applied to the loads for the SILS checks are defined in Table 25. The factor μ must be taken as 1.15 if it refers to the weight of steel elements and 1.25 if it refers to concrete structural elements when the effect of the self weight is considered having an adverse effect, while it is to be taken as 0.95, for both materials, if it has a relieving effect.

In addition to the accidental loads defined in Par. 5.4, it will be necessary to consider the contingency scenario of the failure of one crossbeam and the sections of main longitudinal deck girders connected to it. The analysis shall be done in the dynamic range, considering the sudden detachment of a section of the main deck 60 m long, at the most unfavorable location along the Bridge.

Table 25: Combinations corresponding to Structural Integrity Limit States (SILS).

Combination	PP	PN	QR	QA	VV	VS	VT	A
1	μ	1.0	1.00	-	1.0	-	0.0/1.0	-
2	μ	1.0	1.00	-	-	1.0	0.0/1.0	-
3	μ	1.0	1.00	-	-	-	0.0/1.0	1.0

The intensities of the actions to be included in each combination are determined by multiplying the values defined in the previous Chapter 5 by the factors given in Tables 22, 23, 24 and 25 for the Serviceability Limit States (SLS1, SLS2), Ultimate Limit State (ULS) and Structural Integrity Limit State (SILS) respectively.

7. Characterisation of soils, foundations and anchorage structures

The geotechnical design shall comply with the requirements given in NTC08 [S18]. According to NTC08 [S18] §2.1 the geotechnical design will satisfy safety against ultimate limit state (ULS). This includes evaluation of safety:

1. Against failure mechanisms due to the attainment of bearing capacity and sliding resistance

2. Against instability mechanisms due to attainment of equilibrium conditions

3. Against hydraulic ultimate limit states

Also safety against serviceability limit state (SLS) will be ensured. This includes evaluations of displacements field induced by the relevant load combinations and comparison with threshold values corresponding to the attainment of serviceability limit states.

By virtue of its size, location and intrinsic characteristics, the Bridge has impacts on the territory essentially attributable to the removal, transportation and relocation of large volumes of soil, to transient and permanent changes in ground stresses, and to interaction with the surface and ground water circulation system and to specific effects on ground stability which must be identified and resolved.

The design and construction solutions for the foundations of the towers and the anchor blocks must be characterised, in all phases, by maximum compactness, uniformity and overall simplicity.

The problems of soil-structure interaction must relate both to the average characteristics of the soil and to the possible local variations in composition and mechanical behaviour. Account should be taken of the influence on the overall behaviour of the structural complexity of the ground formations at the various scales of the problem. The size of the foundation structures and of the anchorage blocks will introduce significant changes to the stress state of large volumes of soil. The effects of these changes on the physical and mechanical properties of the soil must be accordingly taken into consideration through time in all construction phases and in the finished structure.

For the foundations, additional ground consolidation works are envisaged, based on the use of jet grouting, which have attributes of structural works and to which the general criteria given in Chapter 8 must be applied.

Structural safety and serviceability of the foundation structures must meet current Italian standards and norms (particularly NTC 08 [S18]).

8. Quality of materials

The materials used in constructing the Bridge and the production and manufacturing technologies must satisfy the basic quality requirements as defined in the other Technical Contract Documents.

The mechanical properties of the materials: resistance to monotonic actions, stress-strain behaviour, strength and behaviour under cyclical loading conditions that cause fatigue, resistance to environmental actions that cause corrosion, must be compatible with the verification levels associated with the return periods specified in Chapters 3 and 4.

These requirements must be proven by means of quality certification and an adequate number of laboratory tests, accepted on the basis of widely accepted quality procedures and guaranteed by means of statistical processes. The fatigue tests shall be carried out for a minimum number of cycles of 10 millions.

The properties of the materials will be used as an input, with an adequate safety margin, in the structural analysis of the Bridge. With regard to these, the possible mechanisms of collapse of the structure or parts thereof must be assessed together with the respective levels of the loading actions.

Choices regarding technologies and materials must be made and documented also in consideration of the specific expertise necessary for the inspection, control and maintenance staff.

9. Devices for controlling the structural response

Passive instruments and devices may be included in the design to guarantee the proper execution of the assembly phases and improvements to the functionality of the crossing with regard to the Serviceability Limit States (SLS1, SLS2). Active devices shall, in case of malfunctioning, at least guarantee sufficient passive performances. In no case their malfunctioning shall compromise the minimum required functionality. For the constituent materials of the devices of this paragraph, reference is made to Chapter 8.

10. Specifications for the construction and assembly phases

The General Contractor must describe the construction methods that he intends to implement, prepare the Method Statements and all necessary specifications for the construction and assembly phase and check structural safety in all construction phases. For particularly critical phases, special equipment and temporary works may be used, although these cannot be relied upon to ensure the safety of the Bridge once complete.

Contingency situations which may occur during temporary structural configurations, associated with the construction process and with the maintenance and dismantling phases, must be identified and assessed, particularly for the definition of residual stress and strain states. The construction process, in each phase, shall not generate geometrical distortions and irreversible deformations in the structural components that may therefore represent hidden defects, i.e. non manifest pathologies, and/or reduce the overall performance capacity of the structure.

Particular attention shall be paid to the prevention of instability phenomena during the construction phase, during which the structure is particularly sensitive to the wind actions.

The General Contractor shall identify and define, assuming full responsibility, the loading actions during the construction works, and the modality of their relative safety checks.

11. Monitoring

In order to guarantee the functionality and safety of the Bridge, a global, integrated and redundant monitoring system must be provided which allows immediate knowledge of the parameters relative to the environment and seismic-tectonic conditions, and of those relative to the traffic (road, rail and marine), to the structural response and to the functionality, safety and durability state of the structure, as defined in detail in Document GCG.F.05.03 Chapter 10.10 “Monitoraggio” (“Monitoring”). For the purposes of monitoring the railway and marine traffic it may also be possible to rely on suitable telecommunication and identification systems.

The information will have to be re-elaborated by a specific calculation unit that will identify and automatically update the structural model according to the actual contingency scenarios. The analyses carried out in real time will have to lead to the definition of significative indexes of reliability that will promptly allow to prepare inspection, maintenance and emergency plans.

The monitoring system shall be designed and implemented together with the structure and will represent an essential element for the management of the structure and the implementation /updating of the management plans, as considered in documents GCG.F.06.01 “Sistema di gestione e controllo” (“Control and Management System”), GCG.F.06.04 “Programma di ispezione e manutenzione” (“Inspection and Maintenance Plan”), GCG.F.06.05 “Programma di esercizio e di gestione delle emergenze” (“Programme of Use and Management of Emergencies”). Various monitoring systems are required for the three different phases of construction, structural testing and operation of the Bridge, which in turn have different scopes and quantities to measure. These will have to be identified before the monitoring system is designed, which shall be optimised in its entirety.

12. References

12.1 ITALIAN LEGISLATION

[L1] Comitato Tecnico Scientifico per il Ponte sullo Stretto di Messina presso il Ministero dei Trasporti e delle Infrastrutture - "Indirizzi progettuali e deliberazioni per il progetto preliminare", 13/12/2002 approvato dal CIPE;

[L2] Voto del Consiglio Superiore dei Lavori Pubblici - Assemblea Generale numero 220/97 dell'adunanza 10/10/97.

12.2 CURRENT ITALIAN STANDARDS FOR DESIGN AND CONSTRUCTION

[S01] Legge 5 Novembre 1971, n.1086 "Norme per la disciplina delle opere di conglomerato cementizio armato, normale e precompresso ed a struttura metallica";

[S02] Legge 2 Febbraio 1974, n.64 – "Provvedimenti per le costruzioni con particolari prescrizioni per le zone sismiche";

[S03] Circolare Ministero dei Lavori Pubblici, 31 Luglio 1979, n.19581 "Legge 5 Novembre 1971, n.1086, Art. 7 – Collaudo statico";

[S04] Circolare Ministero dei Lavori Pubblici, 9 Gennaio 1980, n.20049 "Legge 5 Novembre 1971, n.1086 – Istruzioni relative ai controlli sul conglomerato cementizio adoperato per le strutture in cemento armato";

[S05] Decreto Ministeriale dei Lavori Pubblici, 11 Marzo 1988 "Norme tecniche riguardanti le indagini sui terreni e sulle rocce, la stabilità dei pendii naturali e delle scarpate, i criteri generali e le prescrizioni per la progettazione, l'esecuzione e il collaudo delle opere di sostegno delle terre e delle opere di fondazione";

[S06] Circolare Ministero dei Lavori Pubblici, 24 Settembre 1988, n.30483 "Legge 2 Febbraio 1974, n.64, Art. 1 – DM 11 Marzo 1988. "Norme tecniche riguardanti le indagini sui terreni e sulle rocce, la stabilità dei pendii naturali e delle scarpate, i criteri generali e le prescrizioni per la progettazione, l'esecuzione e il collaudo delle opere di sostegno delle terre e delle opere di fondazione. Istruzioni per l'applicazione";

[S07] Circolare Ministero dei Lavori Pubblici, 20 Luglio 1989, n.1603 “Legge 5 Novembre 1971, n.1086, Art. 20. Autorizzazioni a laboratori per prove sui materiali”;

[S08] Decreto Ministero dei Lavori Pubblici, 4 Maggio 1990 “Aggiornamento delle norme tecniche per la progettazione, la esecuzione e il collaudo dei ponti stradali”;

[S09] Circolare Ministero dei Lavori Pubblici, 25 Febbraio 1991, n.34233 “Legge 2 Febbraio 1974, n.64 - Art. 1 DM 4 Maggio 1990 Istruzioni relative alla normativa tecnica dei ponti stradali”;

[S10] Decreto Ministero dei Lavori Pubblici, 9 Gennaio 1996 “Norme tecniche per il calcolo, l’esecuzione ed il collaudo delle strutture in cemento armato, normale e precompresso e per le strutture metalliche”;

[S11] Decreto Ministero dei Lavori Pubblici, 16 Gennaio 1996 “Norme tecniche relative ai criteri generali per la verifica di sicurezza delle costruzioni e dei carichi e sovraccarichi”;

[S12] Decreto Ministero dei Lavori Pubblici, 16 Gennaio 1996 “Norme tecniche per le costruzioni in zone sismiche”;

[S13] Circolare Ministero dei Lavori Pubblici, 4 Luglio 1996, n.156 aa.gg/stc “Istruzioni per l’applicazione delle “Norme tecniche relative ai criteri generali per la verifica di sicurezza delle costruzioni e dei carichi e sovraccarichi di cui al DM 16 Gennaio 1996”;

[S14] Circolare Ministero dei Lavori Pubblici, 15 Ottobre 1996, n. 252 “Istruzioni per l’applicazione delle “Norme tecniche per il calcolo, l’esecuzione ed il collaudo delle opere in cemento armato normale e precompresso e per le strutture metalliche” di cui al DM 9 Gennaio 1996”;

[S15] Circolare Ministero dei Lavori Pubblici, 10 Aprile 1997, n.65/aa.gg “Istruzioni per l’applicazione delle “Norme tecniche per le costruzioni in zone sismiche” di cui al DM 16 Gennaio 1996”;

[S16] Decreto Ministero dei Lavori Pubblici, 5 Agosto 1999 “Modificazioni al DM 9 Gennaio contenente norme tecniche per il calcolo, l’esecuzione ed il collaudo delle strutture in cemento armato normale e precompresso e per le strutture metalliche”;

[S17] Circolare Ministero dei Lavori Pubblici, 14 Dicembre 1999, n.346/stc “Legge 5 Novembre 1971, n.1086, Art. 20 – Concessione ai laboratori per le prove sui materiali da costruzione”;

[S18] NTC08: DM14.1.2008 Technical construction standard

12.3 INSTRUCTIONS AND QUOTED REFERENCE DOCUMENTS FOR THE DESIGN OF RAILWAY BRIDGES

[R01] Testo RFI n° I/SC/PS-OM/2298 aggiornata al 13/01/1997 "Sovraccarichi per il calcolo dei ponti ferroviari – Istruzione per la progettazione, l'esecuzione ed il collaudo";

[R02] Allegato D – prEN 1991-2 Eurocode 1: Actions on structures – Part 2: Traffic load on bridges.

[R03] Fiche UIC 518;

[R04] Fiche UIC 518 – 1;

[R05] Studi ORE B55/RP8 in appendice a UIC 518;

[R06] Studi ORE C116/RP3 in appendice a UIC 518;

[R07] Studi ORE C138/RP1 in appendice a UIC 518;

[R08] Studi ORE C138/DT66 in appendice a UIC 518.

[R09] RFI 44 F: RFI DTC-ICI-PO SP INF 003 A Fatigue verification of metal railway bridges, 12/10/09

[R10] RFI DTC-ICI-PO SP INF 001 A, dated 12/10/09 ("Superimposed loads for the design of railway bridges – Instructions for design, construction and load testing")

[R11] ORE C138/RP9 in appendice a UIC 518

[R12] Council Directive 96/48/EC of 23 July 1996 on the interoperability of the trans-European high-speed rail system

Appendix 1

Probabilistic properties of the atmospheric turbulence

The atmospheric turbulence consists of three components denominated longitudinal turbulence (u, horizontal, in the direction x of the mean wind velocity), lateral turbulence (v, horizontal, in the direction y orthogonal to the mean wind direction) and vertical turbulence (w, in the vertical direction z). Their probabilistic properties are defined by means of the power spectra, the intensities, the integral scales and the coherence functions defined below.

The power spectra for longitudinal, lateral and vertical turbulences are defined according to frequency (n) and normalised so that the variance is the integral of the spectrum calculated between zero and infinity. These are given in the following relations:

$$\frac{nS_u(z,n)}{I_u^2\bar{u}^2(z)} = \frac{6.868nL_u(z)/\bar{u}(z)}{\left[1+10.302nL_u/\bar{u}(z)\right]^{5/3}}$$

$$\frac{nS_v(z,n)}{I_v^2\bar{u}^2(z)} = \frac{9.434nL_v(z)/\bar{u}(z)}{\left[1+14.15nL_v/\bar{u}(z)\right]^{5/3}}$$

$$\frac{nS_w(z,n)}{I_w^2\bar{u}^2(z)} = \frac{6.103nL_w(z)/\bar{u}(z)}{1+63.181\left[nL_w/\bar{u}(z)\right]^{5/3}}$$

The symbols contained in the above formula have the following meaning:

$\bar{u}$ is the mean wind velocity (defined at par. 5.3.1 of this document)

I_u, I_v & I_w are the intensities of longitudinal, lateral and vertical turbulences respectively (defined as the ratio between turbulence standard deviation and mean wind speed);

L_u, L_v & L_w are the integral scales of longitudinal, lateral and vertical turbulences respectively.

The turbulence intensities are given in the following relations:

$$I_u(z)=\frac{1}{\ln(z/z_0)}\ ;\qquad I_v(z)=0.75\cdot I_u(z)\ ;\qquad I_w(z)=0.50\cdot I_u(z)$$

where z_0 = 0.05 m is the standard roughness length, in relation to wind turbulence.
Given the exceptional length of the Bridge, it is recommended that the change in turbulence intensity along the axis of the deck be assessed and considered in a suitable and justified manner.
The integral scales of turbulence are given in the following relations:

$$L_u(z)=300\cdot\left(\frac{z}{200}\right)^{0.5}\ ;\ L_v=0.25\cdot L_u(z)\ ;\ L_w=0.10\cdot L_u(z)$$

where z is expressed in metres.
The coherence functions for longitudinal, lateral and vertical turbulences are defined as being the ratio between the cross spectrum and the square root of the product of the point spectra and are given in the following relations:

$$Coh_{uu}(M,M',n)=\exp\left\{-\frac{2n\sqrt{c_{ux}^2|x-x'|^2+c_{uy}^2|y-y'|^2+c_{ux}^2|z-z'|^2}}{\bar{u}(z)+\bar{u}(z')}\right\}$$

$$Coh_{vv}(M,M',n)=\exp\left\{-\frac{2n\sqrt{c_{vx}^2|x-x'|^2+c_{vy}^2|y-y'|^2+c_{vx}^2|z-z'|^2}}{\bar{u}(z)+\bar{u}(z')}\right\}$$

$$Coh_{ww}(M,M',n)=\exp\left\{-\frac{2n\sqrt{c_{wx}^2|x-x'|^2+c_{wy}^2|y-y'|^2+c_{wx}^2|z-z'|^2}}{\bar{u}(z)+\bar{u}(z')}\right\}$$

where

M and M' are two coordinate points (x, y, z) and (x', y', z').

The exponential decay factors are given below:

c_{ux}	c_{uy}	c_{uz}	c_{vx}	c_{vy}	c_{vz}	c_{wx}	c_{wy}	c_{wz}
3	10	10	3	6.5	6.5	0.5	6.5	3

Appendix 2

Properties of the real trains RFI

The properties of real trains RFI 1, 2, 3, 4, 5, and 6 are shown here below, in terms of typology, dimensions, and weight.

Characteristics and detailed dynamic schemes of the various railway vehicles to be used in the numerical simulations for railway dynamic analyses (in particular runability) are indicated in the document DT.ISP.F.E.R3.001 "Treni 'reali' di riferimento RFI – Caratteristiche dinamiche per le simulazioni numeriche" ("RFI reference real trains – dynamic characteristics for numerical simulations") included in the "Studies and informative documents".

Train RFI 1 (Type AV -18 t/axle max)

Typical composition: 2 external locomotives + n carriages up to a total length of 400 m

	Box length[m]	Pivot pitch[m]	Axle base [m]	Axle weight [kN]	Nr. of axles	Nr. of bogies
Locomotive	20,0	12,0	3,0	170	4	2
Carriage	26,0	19,0	3,0	106	4	2

Train RFI 2 (Type AV tilting - 14 t/axle max)

Typical composition: sequence of n modules each consisting of 2 locomotives + trailer cars, up to a total length of 400 m

	Box length [m]	Pivot pitch [m]	Axle base [m]	Axle weight [kN]	Nr. of axles	Nr. of bogies
Locomotive	25,0	19,0	2,7	129	4	2
Carriage	25,0	19,0	2,7	120	4	2

Train RFI 3 (Type IC – locomotive with 25 t/axle max and trailer cars with 18 t/axle max)

Typical composition: 1 locomotive + n carriages up to a total length of 600 m

	Box length [m]	Pivot pitch [m]	Axle base [m]	Axle weight [kN]	Nr. of axles	Nr. of bogies
Locomotive	18,9	10,4	2,85	222	4	2
Carriage	26,0	19,0	3,0	180	4	2

Train RFI 4 (Commuting double deck train - 20 t/axle max)

Typical composition: sequence of n modules each consisting of 2 external locomotives + 2 internal trailer cars, up to a total length of 250 m

	Box length [m]	Pivot pitch [m]	Axle base [m]	Axle weight [kN]	Nr. of axles	Nr. of bogis
Locomotive	25,9	18,6	2,7	200	4	2
Carriage	24,1	19,5	2,55	160	4	2

Train RFI 5 (Freight train – locomotive with 25 t/axle max and trailer cars with 25 t/axle max)

Typical composition: 1 locomotive + n carriages up to a total length of 750 m

	Box length [m]	Pivot pitch [m]	Axle base [m]	Axle weight [kN]	Nr. of axles	Nr. of bogies
Locomotive	18,9	10,4	2,85	193.60	4	2
Carriage	12,5	7,5	1,8	250	4	2

Train RFI 6 (Freight train – locomotive with 25 t/axle max and empty trailer cars with distributed mass 1.25 t/m max)

Typical composition: 1 locomotive + n carriages up to a total length of 750 m

	Box length [m]	Pivot pitch [m]	Axle base [m]	Axle weight [kN]	Nr. of axles	Nr. of bogies
Locomotive	18,9	10,4	2,85	193.60	4	2
Carriage	21,7	16,7	1,8	65	4	2