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BIBLIOGRAPHIC REVIEW OF ITALIAN REGULATIONS FROM 1900 TO THE PRESENT FOR THE SIMULATED DESIGN OF ITALIAN RAILWAY BRIDGES

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Abstract

The many catastrophic earthquakes that occurred in Italy in the last fourty years underline how Italy is one of the European countries at the highest seismic risk. For this reason, the vulnerability of infrastructures needs to be investigated. This paper considers the railway network as a case study with a focus on railway bridges, which are neuralgic nodes of the whole system. The most common types of railway bridges in Italy are masonry arch bridges and girder bridges with either a continuous deck or a simply supported deck. Here, we specifically analyze railway girder bridges.

Eucentre evaluates the seismic vulnerability of railway bridges through the definition of fragility curves. Fragility curves result from analytical procedures that determine the seismic behaviour of structures according to the structural typology as well as the level of knowledge of the bridge. One step of these procedures is the simulated design, which aims at reducing the number of variables in the process and requires the knowledge of the regulatory limits used at the time of the design. This paper presents those regulatory limits that concern: (i) loads, (ii) materials resistance, (iii) longitudinal and transversal reinforcement percentages for reinforced concrete bridges, and (iv) bearings. These limits derive from a detailed bibliographic study on Italian regulations, concerning the design of railway bridges from the beginning of 1900 to the present.

Keywords: Railway Bridges, Italian Regulation, Simulated Design, Seismic Vulnerability.

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1 INTRODUCTION

The Italian railway network is made up of about 17000 km of rail lines that are in operation and are largely developed in medium-to-high seismic hazard zones. Important elements of this network are viaducts. The most common types of railway bridges in Italy are masonry arch bridges and girder bridges with either a continuous deck or simply supported deck. In this paper, only the railway girder bridges are analyzed, a brief description of which is also given in relation to the deck material.

The seismic vulnerability of railway bridges is generally assessed through fragility curves. Fragility curves can be obtained through different methods, such as analytical methods that involve the mechanical modeling of the structure.

For the seismic behaviour of the modeled structure to be plausible, the characteristics of materials, the amount of reinforcement, and the types of bearings used in the modeling need to be as close as possible to the actual conditions of the bridge. This information is included in the project documents which are usually not available. To solve this problem, the simulated design of the structure is carried out using the limits imposed by the regulations in force at the time of design.

To this end, this paper provides a brief bibliographic review of the regulations used for the construction of reinforced concrete railway bridges from the beginning of the last century to the present day. The first Italian regulation available on the design of reinforced concrete railway bridges dates back to 1995. For this reason, the provisions on the reinforcement of piers as well as on the mechanical characteristics of materials derive from the regulations issued in Italy starting from 1907 concerning the construction of public reinforced concrete structures. For mobile loads, instead, specific technical standards have been considered starting from 1946.

The definition of the permanent load depends on the composition of the superstructure. For this purpose, this paper also provides the weights of the railway track and ballast, as well as the weight of noise barriers suggested by the latest technical specifications issued by the Italian State Railways ([1], [2], [3]).

Finally, this paper gives a brief bibliographic review of the standards on the types of bearings for railway girder bridges from 1972 onwards. It is very important to identify the geometrical and mechanical characteristics of these devices so that their actual function can be taken into account during their simulated design. A wrong modeling of bearings could affect the actual seismic response of a bridge because it changes the way in which mechanisms of the pier-bearing system are activated.

2 RAILWAY GIRDER BRIDGES

One of a widely used bridge typology in the Italian railway network is represented by girder bridges. The deck can consist of reinforced concrete, prestressed reinforced concrete, or steel, and generally lies on unreinforced concrete, reinforced concrete, or masonry piers. The deck can be simply supported or continuous, thus giving rise to an isostatic or hyperstatic static scheme, respectively. Between the deck and the vertical structure of the bridge are the bearings, the choice of which depends mainly on the type of deck and the type of constraint to be obtained.

In case of a reinforced concrete deck, the method of construction depends mainly on the length of the individual spans as well as on the total length of the bridge. Specifically, the deck can be made of: (i) a slab, (ii) steel girders embedded in concrete, and (iii) reinforced concrete plate girder. The bridge with slab deck is the simplest system used in rail transport, but only when the spans do not exceed 4 m. On the contrary, the deck with steel girders embedded in concrete or reinforced concrete plate girder is used when, for orographic reasons, the deck cannot be very thick. In particular, the deck with steel girders embedded in concrete is no longer than either 13 m, in the case of "double T" beams, or 20 m, in the case of "double T" beams with wide wings. In presence of a deck with reinforced concrete plate girder, instead, the girders also have the function of parapet and can reach heights of also 3.5 m. The deck is continuous and supports the railway superstructure.

In addition to reinforced concrete, the deck can be realized with prestressed reinforced concrete beams. This deck can reach a length that ranges 35 m and 50 m and is generally made of isolated or box beams. When the beam has a box cross-section, the common practice provides for the use of a box for each track. In this case, the beams are connected among each other with cross-beams (which are prestressed, too) to guarantee the beams cooperation in the usual load conditions (e.g. the passage of a train) so that fatigue phenomena can be reduced. Nowadays, the prestressed reinforced concrete deck is really common on both ordinary and high-speed railway lines.

Another type of deck is the steel deck, which has been widely used in the past; nowadays, however, it is mainly used for long span bridges with reduced height or to replace pre-existing steel girders. Because of the high cost of construction and maintenance, steel decks have mostly been replaced by either the prestressed reinforced concrete decks for medium-to-long span bridges, or reinforced concrete and mixed structure decks for bridges with shorter spans. The most common railway steel bridges are made of straight girders lying on masonry or reinforced concrete piers. In the case of bridges with more than one span, however, it is preferable to create non-continuous decks on which the track can be either direct lying or ballasted. Depending on the length of the deck, the direct-laying girders can be twin-girders (for spans between 10-25 m), plate girders (for spans between 30-40 m), or truss girders (for spans between 35-100 m). For what concerns bridges with railway tracks lying on the ballast, they generally have a deck with steel-concrete composite girders. In this case, the beams can be "double T" or box section collaborating with the concrete slab and reach a length of about 20-25 m. To guarantee the interaction between all the girders of the deck, cross-beams are also used, especially in the presence of live loads.

3 ITALIAN REGULATIONS FROM 1900 TO NOWADAYS FOR RAILWAY GIRDER BRIDGES

3.1 Resistance of Materials

The first recommendations about the construction of reinforced concrete structures were issued in Italy with the Royal Decree 10/01/1907 [4]. This decree provided the first indications about the maximum strength to be considered in the structural calculation for both concrete and reinforcement. In particular, this standard stated that the compressive strength of concrete at 28 days of hardening $\sigma_{r,28}$ must not be less than 150 kg/cm^2, the fifth part of which corresponds to the simple compressive safety load of the concrete to be used in the calculation. Concerning the reinforcement, the regulation allowed the use of smooth steel rebars with a tensile strength between 36 kg/mm^2 and 45 kg/mm^2. The safety tensile and shear loads, instead, should not exceed 1000 kg/cm^2 and 800 kg/cm^2, respectively. For the reinforcement, the decree introduced a further limitation, namely the quality coefficient, which is obtained by multiplying the unit failure load per mm^2 by the percentage elongation. This coefficient, which should not exceed 900, was then replaced by the percentage of elongation at maximum tensile strength of the reinforcement in the Royal Decree issued on 4/09/1927 [5]. With the exception of this introduction, the Royal Decree 4/09/1927 [5] has not made substantial changes compared to what was already established by the previous decree [4]. In fact, the decree only reduced and slightly increased the values of the safety concrete load (30 kg/m^2 and 40 kg/m^2 for 2nd and 1st quality concretes) as well as the tensile and shear steel ones (1200 kg/cm^2 and 960 kg/cm^2). As a result, the tensile strength of the steel changed slightly, i.e. 3800-5000 kg/cm^2, corresponding to a minimum elongation of 27% and 21%, respectively.

With the Royal Decree 23/05/1932 [6], the classification of concretes had been improved and divided into Portland, blast furnace, pozzolanic, and aluminous. Portland, blast furnace, and pozzolanic concretes could be of normal or high strength (see Table 1). The mechanical characteristics of steel, instead, did not change compared to what was already indicated in the Royal Decree 4/09/1927 [5].

Mechanical characteristics of concrete	Portland, blast furnace, pozzolanic concrete		Aluminous concrete
	Normal strength	High strength	
Crushing strength [kg/cm^2]	450	600	650
Compressive safety load [kg/cm^2]	40*-50**	50	65
Maximum tangential strength without reinforcement [kg/cm^2]	2	4	4
Maximum tangential strength with reinforcement [kg/cm^2]	14	16	16

*for structures under simple pressure; **for bent structures with a thickness not less than 10 cm.

Table 1: Mechanical characteristics of portland, blast furnace, pozzolanic, and aluminous concretes in the Royal Decree 23/05/1932 [6].

The Circular 17/05/1937 [7] introduced the use of medium carbon steel, only in absence of homogeneous steel. In terms of mechanical characteristics, the medium carbon steel had an ultimate tensile strength between 5000 kg/cm^2 and 6500 kg/cm^2 to which percentages of elongation corresponded to 21% and 14%, respectively. The safety load of the steel increased from 1200 kg/cm^2 to 1600 kg/cm^2.

In 1939 a new Royal Decree [8] was issued to regulate the construction of structures made of unreinforced or reinforced concrete. In this decree, the compressive strength of concrete at 28 days of hardening $\sigma_{r,28}$ had to be at least three times the safety load $\sigma_{c,a}$ adopted in the structural calculations, but never less than 120 kg/cm^2 and 160 kg/cm^2 for normal-strength concrete and high-strength or aluminous concrete, respectively. Regarding the reinforcement, the Royal Decree 16/11/1939 [8] introduced high carbon steel, which allowed to adopt smaller rebar sizes due to its high specific resistance [9].

Table 2 reports the mechanical properties of the three types of steel permitted by the Royal Decree 16/11/1939 [8].

Mechanical characteristics of steel	Mild Steel	Medium carbon steel	High carbon steel
Yield characteristic strength [kg/cm^2]	≥2300	≥2700	≥3100
Ultimate characteristic strength [kg/cm^2]	4200-5000	5000-6000	6000-7000
Elongation [%]	≥20	≥16	≥14
Safety load [kg/cm^2]	≤1400	≤2000	≤2000

Table 2: Mechanical characteristics of steels in the Royal Decree 16/11/1939 [8].

The Royal Decree 16/11/1939 [8] has remained in force until the first years of 1970s as a consequence of a regulatory gap due to the Second World War and the following period of reconstruction. Between 1939 and the early 1970s, however, a series of circulars have been issued, among which it is necessary to mention the Circular 23/05/1957 [10]. This circular not only introduced a new denomination for smooth steel rebars, i.e. AQ42, AQ50 and AQ60, equivalent respectively to the mild, medium carbon, and high carbon steel of Table 2, but also introduced first indications on ribbed steel rebars [9]. The ribbed steel rebars were allowed in special cases, such as shaped or bent rebars, and with a minimum elongation percentage of 12%.

A substantial change with respect to the previous regulations occurred with the Ministerial Decree 30/05/1972 [11] which has laid the foundations of modern legislation. It introduced a classification of ribbed steel rebars as well as a calculation method at the limit states, which replaced the permissible stresses approach. This change defined the transition from a deterministic to a statistical calculation system due to the introduction of the characteristic value. As a consequence, the yield strength and the ultimate strength were not intended anymore as an average value but as the strength that had only a 5% probability of being lowered by the effective resistance [9]. The Ministerial Decree of 30/05/1972 [11] distinguished six classes of concrete by a number that expressed the characteristic cubic resistance at 28 days of hardening R'_{bk}, from which it was possible to obtain the mechanical characteristics of the concrete (e.g. compressive strength, tensile strength, etc.). For what concerns the reinforcement, smooth steels were divided in only two classes (i.e. Fe B 22 and Fe B 32), and the ribbed steel rebars into three classes (i.e. A 38, A 41, and Fe B 44) (see Table 3).

Mechanical characteristics of steel	Smooth steel rebars		Ribbed steel rebars		
	Fe B 22	Fe B 32	A 38	A 41	Fe B 44
Yield characteristic strength [kg/cm^2]	≥ 2200	≥ 3200	≥ 3800	≥ 4100	≥ 4400
Ultimate characteristic strength [kg/cm^2]	≥ 3400	≥ 5000	≥ 4600	≥ 5000	≥ 5500
Elongation [%]	≥ 24	≥ 23	≥ 14	≥ 14	≥ 12
Safety load [kg/cm^2]	1200	1600	2200	2400	2600

Table 3: Mechanical characteristics of smooth steel rebars and ribbed steel rebars in the Ministerial Decree of 30/05/1972 [11].

After 1972, other decrees were issued about the construction of reinforced concrete structures, but they have not considerably changed the provisions of the Ministerial Decree 30/05/1972 [11] in terms of mechanical characteristic of materials. This is evident in Table 4 and Table 5, which report the mechanical characteristics of the rebars defined in the latest standards on the construction of railway bridges, issued by Italian State Railways ([1], [2], [3]). The only significant difference between the Circular n. I/SC/PS-OW2298 1997 [1] and the newest Technical Specifications by Italian State Railways ([2], [3]) is the mechanical characteristics of ribbed steel rebars (see Table 5). As stated in the Italian Seismic Codes (NTC08) [12], the use of smooth steel rebars is definitely prohibited. Concerning concrete properties, no substantial changes were made.

Mechanical characteristics of steel	Smooth steel rebars		Ribbed steel rebars	
	Fe B 22k	Fe B 32k	Fe B 38k	Fe B 44k
Yield characteristic strength [kg/cm^2]	≥ 2150	≥ 3150	≥ 3750	≥ 4300
Ultimate characteristic strength [kg/cm^2]	≥ 3350	≥ 4900	≥ 4500	≥ 5400
Elongation [%]	≥ 24	≥ 23	≥ 14	≥ 12

Table 4: Mechanical characteristics of smooth steel rebars and ribbed steel rebars in the Circular n. I/SC/PS-OW2298 and following update of 1997 [1].

Mechanical characteristics of steel	B450C
Yield characteristic strength [kg/cm^2]	≥ 4500
Ultimate characteristic strength [kg/cm^2]	≥ 5400
Elongation [%]	≥ 7.5

Table 5: Mechanical characteristics of ribbed steel rebars in the newest technical specifications by Italian State Railways ([2], [3]).

3.2 Definition of loads

3.2.1 Permanent loads

The permanent loads are obtained from the structural and non-structural loads, to which the hydraulic and lateral earth pressures are added too, if existing.

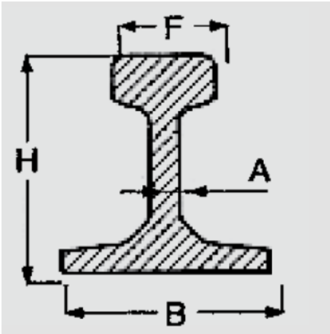
The permanent structural loads are evaluated on the basis of the geometric characteristics of the elements constituting the deck of the bridge (i.e. girders, cross-beams, and slab) and the specific weights of the materials with which they are made. The latter are regulated by the codes in force at the time of the structure design.

The non-structural load, instead, is obtained by adding the weight of all non-structural elements composing the superstructure, that are mainly: (i) the railway track, (ii) the ballast, (iii) the waterproofing, and (iv) the noise barriers.

A railway track is defined as the totality of rails, sleepers, and fasteners.

The first rails were entirely made of cast iron, were very short, and mounted on large stone blocks drowned in the ground. Due to the fragility of cast iron, it was later replaced by iron, which is, however, subjected to wear. The solution to the problem of wear was the “double headed rail”, which was mounted through hardwood wedges into special fixing plates, fixed in turn on wooden sleepers. When the upper table was worn out, the rail was inverted so that the lower table was ready to be used. As this process was complex and expensive, other solutions were developed, such as the “Vignoles rail” (named by the engineer who invented it), which has a “double-T” profile with a wider lower table than the upper one. Advances in material science led to the replacement of iron with steel: steel performs better than iron and deforms less at high speeds. The intended use of the tracks defines the type of steel used for the rails, their dimensions and, consequently, the weight and maximum permissible speed of the vehicles. The higher the linear weight of the rail, the higher are its performances. Initially, the rails had a weight of 18-25 kg/m or 27 kg/m. Later on, rails with a weight of 36 kg/m and 46-48 kg/m became very common for secondary and main railways, respectively.

Since the 1960s, during the maintenance of existing railway systems, Italian State Railways replaced the old rails with “Vignoles” types UIC 50 and UIC 60. In Italy, the code that describes the “Vignoles” type rails is UNI-3134 [13] which doesn't differ much from the European and UIC (i.e. Union Internationale des Chemins de fer) regulations. Table 6 shows the weight of rails in kg/ml according to UNI-3141 [13].



Type	Weight [kg/m]	H [mm]	F [mm]	B [mm]	A [mm]
21	21.373	100	50	80	10
27	27.350	120	50	95	11
30	30.152	125	56	100	12
36	36.188	130	60	100	14
46	46.786	145	63.5/67.2	135	14
50	49.850	148	65.2/70	135	14
60	60.34	172	70.6/74.3	150	16.5

Table 6: Weight and geometrical properties of rails in kg/m according to UNI-3141 [13].

The sleepers, instead, can be made of wood or prestressed reinforced concrete.

Wooden sleepers are the oldest and still widespread. For reasons of cost, of supply as well as of a reduced service life due to adverse weather conditions, in the past wooden sleepers were replaced by iron ones. Nowadays, especially on high-speed rails, prestressed concrete mono-block or twin-block sleepers are used. The twin-block sleepers differ from the mono-block types because they consist of two parallelepiped prestressed concrete elements connected to each other by a steel bar.

Table 7 shows the geometric characteristics and the sleeper's weight in kilogram. To calculate the weight per meter, it is necessary to refer to the module, i.e. the number of sleepers that are in 6 m of track length. The module depends on the class of the railway line, and consequently on the maximum load and speed of the train, and can be:

- *Module 6/10*, i.e. 10 sleepers in 6 m (used for tracks belonging to the main rail network suitable for the highest speeds, for 18 t/axle in case of isolated axles, and for more than 18 t in case of locomotive axles);
- *Module 6/9*, i.e. 9 sleepers in 6 m (used for complementary rail network as well as for crossing or priority rails suitable for lower speeds, for 18 t/axis in case of isolated axles, and for more than 18 t in case of locomotive axles); and
- *Module 6/8*, i.e. 8 sleepers in 6 m (used for tracks belonging to the secondary rail network as well as for all other rails suitable for even lower speeds and 16 t/axis in case of isolated axles).

To connect the sleepers to the rails, and often to ensure electrical insulation, the used fasteners can be of different types; their weight has little impact on the non-structural load.

Type of sleeper	Length [mm]	Depth [mm]	Thickness [mm]	Weight [kg]
Wood	2600	260	160	80 – 100
Prestressed concrete mono-block	2300 - 2600	300	180 – 230	250 – 370
Prestressed concrete twin-block	2300	300	220	245

Table 7: Geometric characteristic of the sleepers.

The railway tracks generally lay on a layer of ballast; the latter can be replaced by a concrete plate in very special cases, such as viaducts with an isostatic structure and length span not exceeding 30 m. The ballast is a layer of crushed stone with a thickness that can be about 50 cm or 35 cm, depending on whether the importance of the rail network is primary or secondary. The stone material has an internal friction angle of not less than 45° and an apparent volumetric mass of not less than 1.5 t/m^3.

Between the deck and the ballast exists a waterproofing layer consisting of two sheaths, one at the top and one at the bottom, and a protective layer of bituminous conglomerate about 5 cm thick. The membranes at the top and the bottom of the waterproofing layer have a thickness of about 5 mm and 3 mm, respectively, and a weight of 4 kg/m^2 and 3-3.5 kg/m^2, respectively. The bituminous conglomerate generally weighs 80-90 kg/cm^2.

Finally, the superstructure is also composed of noise barriers, the weight of which is usually taken from the manufacturer's data sheets.

Table 8 shows the weight and height to be considered for the design of the noise barriers provided by: (i) the Circular n. I/SC/PS-OW2298 1997 [1], (ii) the Technical Specification "RFI DTC INC PO SP IFS 001 A" 2011 [2], (iii) and the Manual for the Design of Civil Structures "RFI DTC SI PS MA IFS 001 A" 2016 [3].

Regulation	Weight [kN/m^2]	Height from the slab floor [m]
Circular 13/01/1997 [1]	2,0	4,0
Technical Specification "RFI-DTC-INC-PO-SP-IFS-001-A" 2011 [2]	4,0	4,0
Manual for the design of civil Structures "RFI DTC SI PS MA IFS 001 A 2016 [3]	4,0	4,0

Table 8: Prescriptions for noise barriers ([1], [2], [3]).

In the case of a superstructure consisting of ballast, railway track, and waterproofing (including the bituminous conglomerate), the non-structural load can be simply calculated by considering a total volume weight of 18 kN/m^3. This volume weight is applied over the entire average width between the paraballast walls, for an average height between the top rail and the extrados of the deck equal to 0.80 m. For bridges in curved railway sections, , the weight of the ballast used to create the superelevation must be added to the conventional weight indicated above, which is evaluated with its actual geometric distribution and with a volume weight of 20 kN/m^3 ([1], [2], [3]).

3.2.2 Train Live Loads

The simulated design of railway bridges requires an estimate of the train live loads used in the planning step according to the year of structure design.

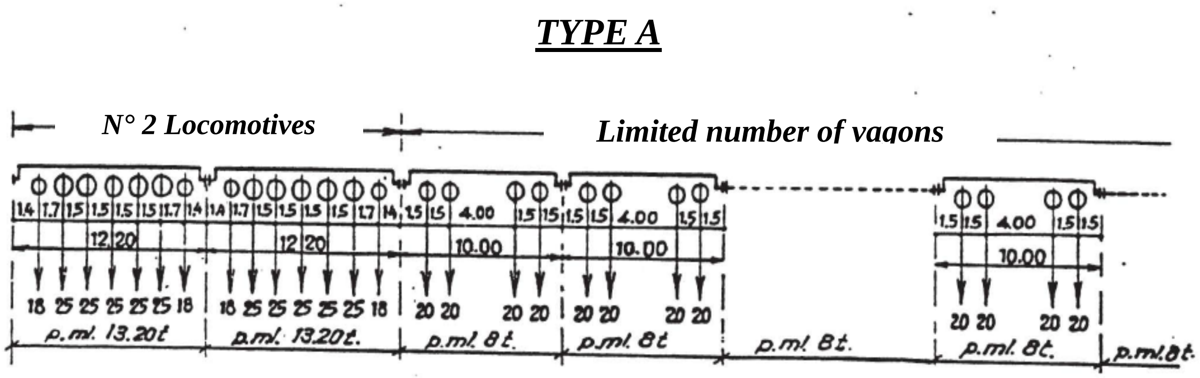
For reinforced concrete bridges, the first regulation on this topic was issued in 1946, when Circular 30/10/1946 [14] introduced a formula for the evaluation of a dynamic increase coefficient φ to be used for reinforced concrete bridges:

$$\varphi = \frac{0.4}{1+0.2L} + \frac{0.6}{1+4P/S}, \tag{1}$$

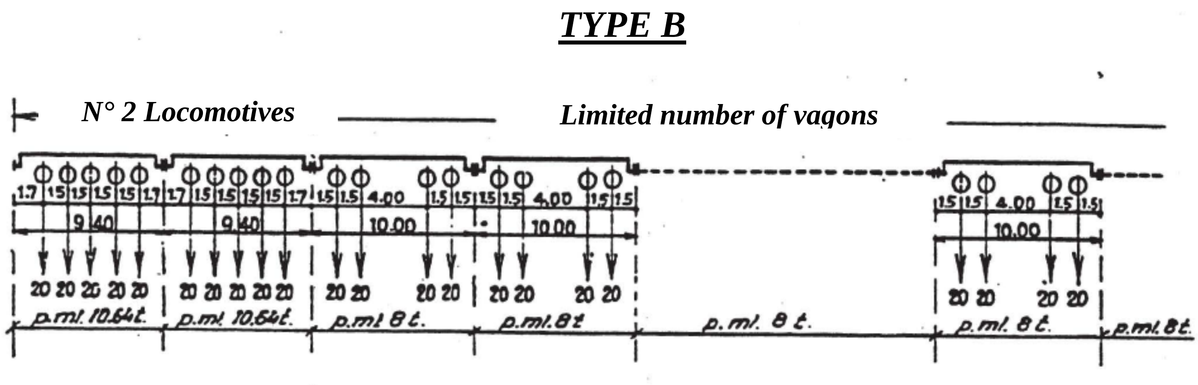
in which L is the theoretical load capacity of the beam, P the overload, and S the permanent load. For the concrete structures design before the Circular n. I/SC/PS-OW2298 1997 [1], the increase in load due to the passage of trains was carried out by increasing the values of overloading by 25%. This criterion derived from some regulations concerning the construction of concrete structures that were issued before 1945.

From 1945 until the following year, instead, the design of reinforced concrete bridges was carried out by using the dynamic increase coefficient φ defined in the Circular 15/07/1945 [15] for steel bridges: its value depended on both the design speed V and the theoretical capacity L of the structural element to be designed. Specifically, if rail joints were absent or only welded, the dynamic increase coefficient φ was assumed to be maximally of 40% to be associated to the elements of limited theoretical capacity L. This value was subsequently increased to 50% as well as the maximum design speed V_{max} that became equal to 100 km/h. As a consequence, the dynamic increase coefficient φ never exceeded 50%. Concerning the train live loads, Circular 15/07/1945 [15] defined two types of load in relation to the presumed railway traffic, i.e. “Type A” and “Type B” (see Figure 1). Alternatively, for the calculation of longitudinal ties, cross-beams, and structural elements with a low theoretical load capacity, the standard allowed the use of an isolated axle of 30 t and 25 t, instead of the train live loads “Type A” and “Type B”, respectively.

Before the Circular 15/07/1945 [15], other regulations defining the train live loads were issued in Italy, but we did not consider them here as they concerned exclusively iron and masonry railway bridges, which were in the past much more common than the ones made of reinforced concrete. In Figure 2 we report only the train live loads used to design railway bridges before 1916, when the Italian State Railways was founded.

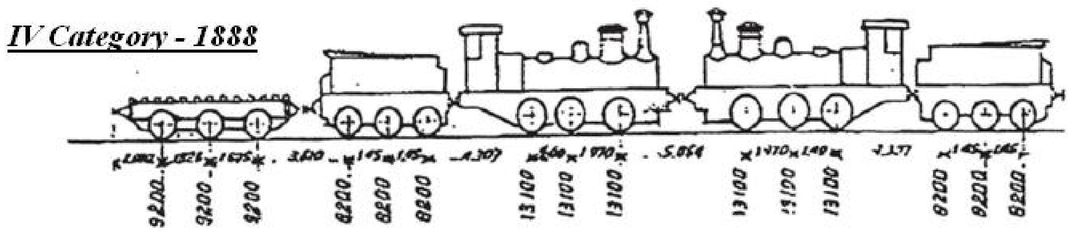


(a)

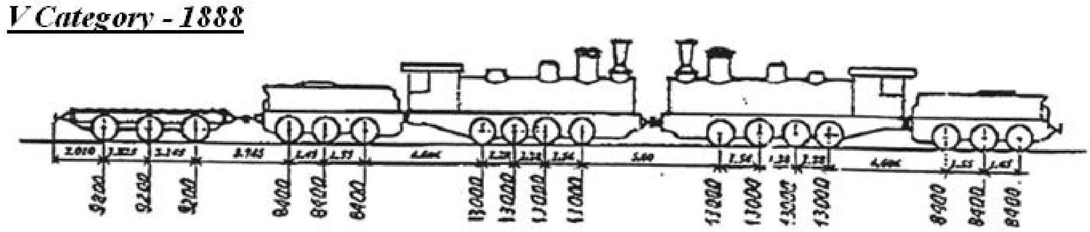


(b)

Figure 1: (a) “Type A” and (b) “Type B” train live loads, used for the design of railway bridges according to Circular 15/07/1945 [15].



(a)



(b)

Figure 2: (a) "Category IV" and (b) "Category V" train live loads, used to design railway bridges before the standard issued in 1916.

The provisions of the Circular 15/07/1945 [15] have remained valid until 1995, when the Circular n. I/SC/PS-OW2298 *"Overloads for the calculation of railway bridges - Instructions for design, execution and testing"* were issued, and then updated in 1997 [1]. This Circular incorporated the instructions from the Eurocodes and introduced two new models of train live load that replaced the previous ones. The new models of train live loads were:

- for normal railway traffic "LM71";
- for heavy railway traffic "SW", which was available in two different configurations "SW1" and "SW2".

The characteristic values attributed to the load models had to be multiplied by an adaptation coefficient α, which varied according to the category of the railway bridge to be designed (see Table 9). Bridges could belong to two different railway categories, i.e. " Category A" and "Category B". In particular, bridges in "Category A" applied to railways on which heaviest trains circulated or should be able to circulate; on the contrary, bridges in "Category B" applied to the secondary railways of normal gauge, not included in "Category A".

Load model	Adaption coefficient α	
	Category A	Category B
LM71	1.1	0.83
SW/0	1.1	0.83
SW/2	1.0	0.83

Table 9: Adaptation coefficient α depending on the load model and the bridge category defined in Circular n. I/SC/PS-OW2298 " and its following update [1].

As it can be seen from Figure 3a, contrary to the provisions of the previous regulations, the train live load was represented by both axial and distributed loads.

In particular, the load model for normal traffic "LM71" consisted of four axles Q_{vk} of 250 kN, placed at a distance of 1.60 m from one another, and of a distributed load q_{vk} of 80 kN/m in both directions, starting from 0.8 m from the end axles Q_{vk} till an unlimited length. In addition, for this load model an eccentricity of the load towards the track axis was contemplated, depending on the gauge *s*, to take into account the displacement of loads.

The load model for the heavy traffic "SW" (see Figure 3b), instead, was characterized by two distinct configurations, as shown in Table 10.

Finally, for some particular controls, the Circular introduced a special train live load called "Unloading Train", represented by an uniformly distributed load equal to 12.5 kN/m.

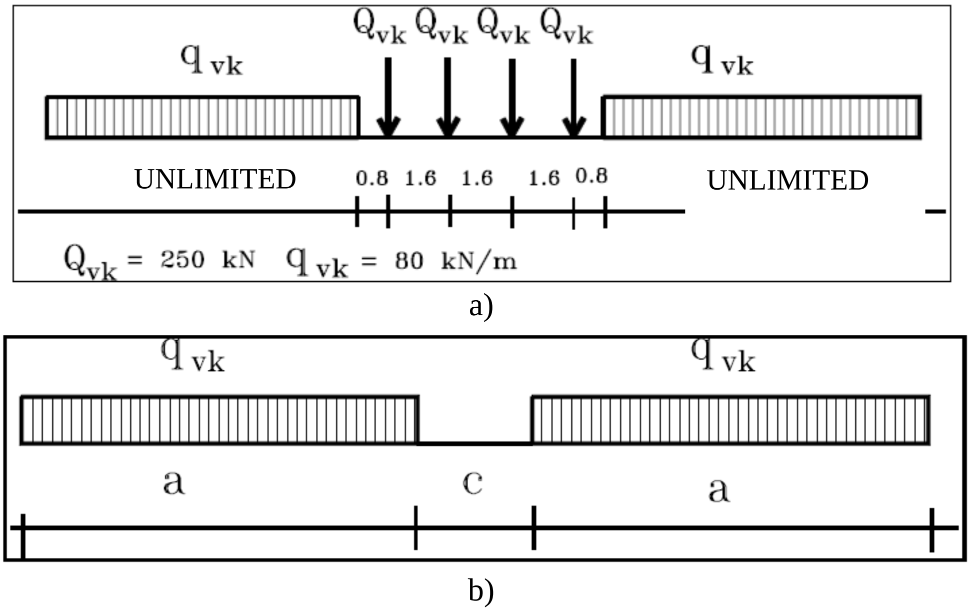


Figure 3: a) Load model for normal railway traffic "LM71" and b) Load model for heavy railway traffic "SW" defined in the Circular n. I/SC/PS-OW2298 and following update in 1997 [1].

Load model	Q_{vk} [kN/m]	a [m]	c [m]
SW/0	133	15.0	5.3
SW/2	150	25.0	7.0

Table 10: Load model for heavy railway traffic "SW" defined in the Circular n. I/SC/PS-OW2298 and following update in 1997 [1].

The Circular n. I/SC/PS-OW2298 1997 [1] further introduced two different dynamic increase coefficients φ in relation to the state of maintenance of the railway line which could be standard or reduced. These coefficients had to be applied to the theoretical load models "LM71" and "SW" in case of a static analysis, to take into account the movement effects and any imperfections of the rail, the wheels, and the suspension system. The Circular n. I/SC/PS-OW2298 1997 [1] also contemplated real dynamic increase coefficients φ_{real} to be multiplied to load models representing a real train. Real train load models are used in particular conditions of analysis (or for the design of specific bridge typologies) and are made of concentrated loads, variously spaced, that schematize the succession of the axes of convoys actually or potentially circulating; each of them was characterized by a maximum speed and a certain overall length [3].

Finally, the latest "RFI DTC INC PO SP IFS 001 A" 2011 [2] and "RFI DTC SI PS MA IFS 001 A" 2016 [3] assume the same load models defined in the Circular n. I/SC/PS-OW2298 1997 [1]. The only differences are: (i) the adaptation coefficient α, which herein depends only on the load model (see Table 11), (ii) the calculation of the real dynamic coefficient φ_{real}, and (iii) the value corresponding to the unloading train, which became 10.0 kN/m.

Load model	Adaption coefficient α
LM71	1.1
SW/0	1.1
SW/2	1.0

Table 11: Adaptation coefficient α depending on the load model defined in “RFI DTC INC PO SP IFS 001 A” 2011 [2] and in “RFI DTC SI PS MA IFS 001 A” 2016 [3].

3.3 Longitudinal and transverse reinforcement

In the seismic assessment of existing reinforced concrete bridges through analytical methods, the amount of longitudinal and transverse reinforcement in the piers needs to be known. This information is usually given in construction plans, but not often available. To fill this gap, the amount of the reinforcement can be derived from the appropriate code that was in use at the time of the structure design.

Table 12 reports the minimum values of both longitudinal and transverse reinforcement defined from the Royal Decree 16th November 1939 [8] to the Ministerial Decree 4th 1992 [16]. The Royal Decree 10th January 1907 [4] is excluded since it does not contain any information on the reinforcement to be placed in the reinforce concrete structural vertical elements. Instead, we deal separately with the Circular n. I/SC/PS-OW2298 1997 [1], the Technical Specification “RFI DTC INC PO SP IFS 001 A” 2011 [2], and the Manual for the Design of Civil Structures “RFI DTC SI PS MA IFS 001 A” 2016 [3] because they are regulations issued directly by Italian State Railways.

In particular, for all viaducts built in territories seismically classified in 1st category (S=12, where *S* represents the seismicity level), 2nd category (S=9), 3rd category (S=6), the Circular n. I/SC/PS-OW2298 1997 [1] defined the minimum quantity of longitudinal reinforcement as 0.6%A_{eff}, where A_{eff} is the area of the effective concrete cross-section. This limit changed to 0.4%A_{eff} for all viaducts designed in the 4th seismic zone category (i.e. not classified zone), with the exception of the Sardinia region. In addition, the longitudinal rebars had not to be more than 300 mm apart. The diameter of the stirrups and transverse ligatures, instead, had not to be less than 8 mm and was obtained according to relationships that depended on the cross-section type of the pier, i.e. whether they were rectangular or circular, in both cases solid or hollow. Furthermore, in piers with a solid or hollow circular cross-section the use of spirals was not permitted anymore and were replaced by circular stirrups. To increase the level of ductility of the structure, the distance *s* between the stirrups had to be more than 10 times the minimum diameter of the vertical rebars.

The Technical Specification “RFI DTC INC PO SP IFS 001 A” 2011 [2] and the Manual for the Design of Civil Structures “RFI DTC SI PS MA IFS 001 A” 2016 [3] resumed with the established values from the Circular n. I/SC/PS-OW2298 1997 [1]. Regardless of the seismic category for which the viaduct was designed, both Technical Specification and Manual for the Design of Civil Structures ([2], [3]) fix a minimum longitudinal reinforcement value of 0.6%A_{eff}. As established in the Circular n. I/SC/PS-OW2298 1997 [1], the longitudinal rebars should be connected by stirrups, which should have a diameter greater than 8 mm, but a pitch neither greater than 10 times the diameter of the longitudinal bars they connect nor 1/5 of the diameter of the section core inside them. In addition, piers with hollow section should have at least 6 ties per square meter connecting the longitudinal reinforcements. Finally, the limitations to define the diameter of pier stirrups provided by the Circular n. I/SC/PS-OW2298 1997 [1] were also present in the Technical Specification [2] and the Manual for the Design of Civil Structures [3]. The only difference was that such limitations only applied if the bridge design required the use of a structure factor *q* equal or less than 1.5.

Code	Longitudinal reinforcement	Transversal reinforcement
R.M.D. 16/11/1939 [8]	$A_s \geq 0.8\ \%\ A_c$ ($A_c \leq 2000$ cm^2) $A_s \geq 0.5\ \%\ A_c$ ($A_c \geq 8000$ cm^2) $0.8\%A_c<A_s<0.5\%A_c(2000cm^2<A_c<8000cm^2)$	$p \leq min(1/2L_{min},10\phi_l)$
M.D. 30/05/1972 [11]	$0.6\ \%A_c \leq A_s \leq 5\ \%A_c$ $A_s \geq 0.3\ \%\ A_{eff}$ $\phi_l \geq 12$ mm	$p \leq min(15\phi_{l,min};25cm)$ $\phi_t \geq 6$ mm
M.D. 30/05/1974 [17] M.D. 16/06/1976 [18]	$A_s \geq 0.6\%A_c$ $0.3\%A_{eff} \leq A_s \leq 5\%A_{eff}$ $\phi_l \geq 12$ mm	$p \leq min(15\phi_{l,min};25cm)$ $\phi_t \geq 6$ mm
M.D. 26/03/1980 [19] M.D. 01/04/1983 [20] M.D. 27/07/1985 [21] M.D. 14/02/1992 [16]	$A_s \geq 0.8\%\ A_c$ $0.3\%A_{eff} \leq A_s \leq 6\%A_{eff}$ $\phi_l \geq 12$ mm	$p \leq min(15\phi_{l,min};25$ cm) $\phi_t \geq max(6mm;1/4\phi_{l,max})$

A_s=Total steel area in a reinforced concrete pier cross-section; A_c=Concrete area strictly necessary for the axial load; ϕ_l=Diameter of the longitudinal steel in the pier cross-section; A_{eff}=Effective concrete cross-section; p=Stirrups pitch; ϕ_t=Stirrups diameter; L_{min}=Smallest size of the pier cross-section; $\phi_{l,min}$, $\phi_{l,max}$ =Smallest and biggest diameter of the longitudinal steel in the pier cross-section.

Table 12: Longitudinal and transverse reinforcement of reinforced concrete piers from the Royal Ministerial Decree 16th November 1939 [8] to Ministerial Decree 4th February 1992 [16].

3.4 Bearings

Another necessary specification for the seismic verification of existing bridges relates to the actual function of supports in the structural response. The structural response cannot properly model the reality in case of inadequate modelling of support devices.

To this end, this section contains a brief description of the state of the art of bearings available in the Italian regulations from 1972 to the present.

The Technical Instructions CNR 10018/72 [22] provided guidance for the calculation, acceptance, and installation of elastomeric bearings, which could also be reinforced. To design this type of bearing required to define: (i) the normal force due to vertical loads acting on the support surface, (ii) the horizontal force due to permanent loads and thermal variations, and (iii) the horizontal force depending only on accidental loads and maximum rotation expected in the beam plane. Considering the most unfavorable combinations of the above mentioned loads and deformations, it was possible to calculate: (i) the normal tension due to the vertical load, (ii) the tangential tensions produced by the vertical load, the horizontal loads and the rotation, and (iii) the shortening and the elastic sliding due to the vertical load and the horizontal forces, respectively. These stress values were then used to carry out strength, sliding, and stability analyses in case of elastomeric bearings. In case of reinforced elastomeric devices, the verification of reinforcement was added to the analyses [23].

In 1985, Technical Instructions CNR 10018/72 [22] was updated. Specifically, in addition to elastomeric devices, the Technical Instructions CNR 10018/85 [24] allowed also the use of confined elastomeric disc bearings as well as PTFE slide bearings (PTFE stands for polytetrafluoroethylene, commonly known as teflon).

The confined elastomeric disc bearings allow rotations around any horizontal axis due to the deformability of an unreinforced rubber disc, confined within a steel monolithic base.

PTFE slide bearings, on the other hand, use sliding surfaces that consist of a layer of PTFE sliding on a surface with a flat, cylindrical, or spherical shape and that is usually made of stainless steel or aluminum alloy. To increase the load resistance, teflon is often combined with other materials (e.g. graphite, glass fibers, etc.), which can bring its load resistance up to

1000 daN/cm^2. Although they have mostly good properties, the negative characteristic of these bearings is that they deplete due to their highly viscous behavior, especially if subjected to reciprocal translations [25].

In 1987 the Technical Instructions CNR 10018 instructions were updated again: in this case the changes mainly concerned the level of the maximum horizontal forces that the supports could transmit to the structure [26]. In particular, bearings with metal plates solidified with reinforced concrete structures with cement or epoxy malt could transfer shear stresses of no more than 25% of the normal concurrent action. For supports with ribbed steel plates as well as with appropriate cavities or prominences, this limit could be increased by up to 35%. For shear stresses exceeding the above limits, mechanical connections had to be used (e.g. pins, clamps, etc.), especially in presence of bridges located in zones of high seismic hazard or when potentially exposed to significant dynamic actions [23].

In 1996, the Technical Instructions FS 44/B [27] were issued and incorporated all the innovations introduced by the Decree of the Ministry of Public Works 16.1.96 [28] in case of under-track structures designed in zones of high seismic hazard.

In 1998, the Technical Instructions CNR 10018 were improved again [29]. An important change compared to the latest version of the instructions, i.e. CNR 10018/85 [24], was that the use of natural rubber or neoprene devices was prohibited in favor of reinforced elastomeric bearings. Initially, natural rubber or neoprene devices were considered a valid alternative to steel devices, characterized by phenomena such as slippage (for cylindrical steel devices) or corrosion (both punctual and uniform). However, they were abolished because they showed several problems, such as deterioration due to atmospheric agents, hardening of the rubber, sliding of the support, as well as phenomena of fatigue and creep. These negative aspects were greatly reduced in reinforced elastomeric devices, which were capable of withstanding simultaneous loads and displacements in any direction and of restoring the undeformed configuration after the permitted displacements. Moreover, being immersed in the rubber, the steel of the reinforced elastomeric bearings is preserved from the phenomenon of corrosion. The only phenomenon that can cause the collapse of such devices is the delamination between elastomer and steel, which, depending on the entity, can cause the total or partial loss of compressive strength without additional deformation. Finally, for the design and verification of the elastomeric devices, the Technical Instructions CNR 10018/85 [24] considered the actions to which they are subjected as well as the permitted displacements, both of which had to be calculated using a three-dimensional analysis of the structure.

In addition to the different types of bearings already described in the previous versions, the Technical Instructions CNR 10018/98 [29] provided new information on bridge bearings made entirely of steel. The functioning of the steel devices is based on the physical rolling phenomenon of two or more steel surfaces in contact with each other.

In 2002 the Technical Instructions FS 44/E were issued [30] that initially defined and described in detail which bearings had to be used for railway bridges, also according to the type of movement allowed in the plane. In particular, the bearings could be fixed or movable. If they are fixed, all the translations are prevented, otherwise translations in one or more directions of the plane are allowed. The Technical Instruction FS 44/E [30] also stated that all bearings had to be capable of rotation in accordance with the values given in the instruction itself and that, if bearings were required to transmit high horizontal forces, it might be convenient to use seismic restraints (fixed or movable).

Finally, the recommendations defined in the Technical Instructions FS 44/E [30] were also included in the Technical Specification "RFI DTC INC PO SP IFS 005 A" 2011 [2], and then later replaced by the Manual for the Design of civil Structures “RFI DTC SI PS MA IFS 001 A” 2016 [3]. The latter provides the requirements for the calculation, construction, installation, and control of bearings used for railway bridges in accordance with the New Technical Regulations [12], and following the Ministerial Circular 2 February 2009 no. 617 [31].

4 CONCLUSIONS

This paper gave a bibliographic review about the regulations on the design of railway girder bridges, issued in Italy from 1900 to the present.

In particular, the paper intended to report the main changes that have occurred at the regulatory level on the construction of reinforced concrete railway bridges, especially in terms of mechanical characteristics of materials, loads, longitudinal and transverse reinforcements, as well as bearings.

The first Italian regulation on the design of reinforced concrete railway girder bridges that is publicly available dates back to 1995. The provisions on the reinforcement of piers and on the mechanical characteristics of materials derived from regulations that had been issued in Italy starting from 1907. For mobile loads and bearings, technical standards specifically issued for railway bridges had been taken into account from 1946 until 1972.

The definition of the permanent load depends on the composition of the superstructure. For this purpose, this paper also provided information about the weights of the railway track and ballast, as well as the weight of noise barriers.

The bibliographic review of the Italian regulations given in this paper provides a collection of data that is necessary for the simulated design of railway reinforced concrete girder bridges in Italy using analytical methods. The presented data depend on the year of design and the type of railway bridge and eventually define the likely seismic behavior of these structures.

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