Bridges for High-Speed Railways

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Preface

The implementation of the high-speed railway network constitutes, nowadays, one of the most important European challenges both from the social/economic and technical point of view. One of these challenges results from the trains travelling at high speeds, which produce new and complex effects, namely on bridges, and impose a more accurate design and new construction technologies.

Many countries have already developed their own structural solutions for dealing with the effects of high-speed trains in bridges, and a great number of structures have been in operation for several years. However, in recent years, design concepts and technology have improved, while innovative structural ideas have appeared.

In this context, this book includes the contributions of a group of international specialists in this field, sharing their knowledge and expertise with the engineering community, and discussing the structural behaviour and performance of existing solutions and potential improvements.

The book includes a number of chapters covering the following topics:

– Design;
– Codes and Dynamic Analysis;
– Construction;
– Monitoring, Maintenance and Repair.

The themes included in this book are mainly based on the papers presented at the workshop "BRIDGES FOR HIGH-SPEED RAILWAYS" organised by the Faculdade de Engenharia da Universidade do Porto (FEUP). This book will be completed with two others, involving more focused topics: "Dynamics of High-Speed Railway Bridges" and "Track-Bridge Interaction on High-Speed Railways".

The editors would like to thank all those who contributed to this book, in particular our distinguished guest chapters' authors who heightened, with their knowledge and expertise, the present interest and quality of the book, the support of the sponsors for the events which originated the materials for this book, and the institutional support of the Faculty of Engineering of the University of Porto and the RAVE – Rede Ferroviária de Alta Velocidade, S.A.

We hope this book will be helpful not only to those professionals involved in the design, construction, or maintenance of high-speed railway systems, but also to researchers and students working in this field.

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CHAPTER 1

Steel and composite bridges for high speed railways – the French know-how

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1 INTRODUCTION

Nowadays in France, above 80% of bridges in the medium span range have a composite steel concrete deck. This trend started in the 1980's in the road bridges. Before that in the 1970's the development of pre-stressed concrete bridges in France had totally excluded steel from the bridge market.

About ten years later, the same phenomenon was observed in the rail bridges for the high-speed lines; (Table 1). The first two lines in the 1990's had only pre-stressed concrete viaducts, but the share of steel has gradually been increasing since that time.

This paper explains how steel and composite steel concrete structures gradually became the most widespread type in the large bridges of the high-speed railway lines in France. It gives also some examples of composite bridges for high speed rail lines abroad, using the French know-how.

2 THE COME–BACK OF STEEL IN BRIDGES

Steel for bridges is not an important market in France, the annual structural steel weight in this construction branch bridges varies between 30 and 40 thousand tonnes. Bridges however provide a highly mediatic image for steel and the steel industry is keen to promote this image. Although

Table 1. Steel in TGV bridges.

High speed railway line	Steel weight	Year
TGV SUD EST	xxxx	1983
TGV Atlantique	xxxx	1990
TGV NORD	9573 T	1993
LILLE	3350 T	1996
Interconnexion	4220 T	1996
Rhone Alpes	3595 T	1994
Total partiel 1990–1994	20738 T	
TGV Mediterannee	42475 T	1999
TGV EST	25000 T	2005

Table 2. Steel bridge types.

Bridge type	N°	Deck length	%Total steel
Tied-Arch	3	443 m	24
Truss girder	1	300 m	3
Twin girders	17	7370 m	57
Four girders	8	500 m	9
Twin boxes	3	370 m	2
Lateral girders	3	245 m	5

the total weight remains nearly the same, the percentage of deck surface in the medium and large bridges has been constantly rising since the 1970's. In these last years, major bridge projects are being built with steel both for motorways and high-speed railways.

This comeback is due to the special research and development efforts of all the professionals involved in the steel bridges: owners, designers, builders, steel fabricators. The most important of these relate to the design of uncomplicated structures, the use of modern calculation rules based on the limit states concept and the availability of high quality thick steel plates.

The French Steel Bridges Committee has also played an important role in the development of the steel bridge market. Since the early 1980s, for large and medium bridges a double tender has been made as often as possible, with two alternatives: prestressed concrete and steel-concrete composite deck. Professional training programmes have also been organised regularly to familiarise the design engineers with steel and composite bridges.

Statistics of road bridges shows that a single type of structure accounts for the success of steel bridges: the twin girder composite bridge. This structural type accounts for over 80% of new bridges. The economic advantage of the twin girder deck was first demonstrated in the motorway bridges. For this reason the French Railways with the help of the steel bridges constructors and the steel fabricators carried research programs and proved the competitiveness of these decks for the new railway bridges.

This comeback took place in the new TGV railway bridges only after 1990. The first high speed railways in the eighties, the South East and Atlantic lines, had only pre-stressed concrete viaducts while the North TGV line from Paris to Lille included some fifteen steel and composite bridges for a total weight of 20000 tons.

This trend was confirmed in the new Mediterranean TGV with 44000 tons of steel in 23 bridges, counting for nearly two thirds of the total large crossings. This exceeds the current annual steel consumption in French bridges which reaches about 30000 tons.

About 42000 tons of thick plates were furnished by the factories of Dillinger Hütte GTS at Dillinghen in Germany and Dunkeque in France.

3 THE STRUCTURAL DESIGN

In the TGV Mediterranean line, the seven largest viaducts were designed with an architectural competition; among these four had a steel structure.

Steel has played an essential role by enabling the high speed crossing of spans ranging from 25 m to over 100 m. For the longest spans, tied-arch bridges were retained; for the shorter spans plate girders decks were chosen: composite two or four beams, twin boxes or lateral girders. Table 2 shows the structural types with the total length and the percentage of steel consumption.

The twin girder composite deck proved its highly competitive and economic value throughout the line. The design was optimised through easy to fabricate structural details especially for the lower bracing. In some bridges this bracing was replaced by concrete slabs, which were even more economical (Fig. 1).

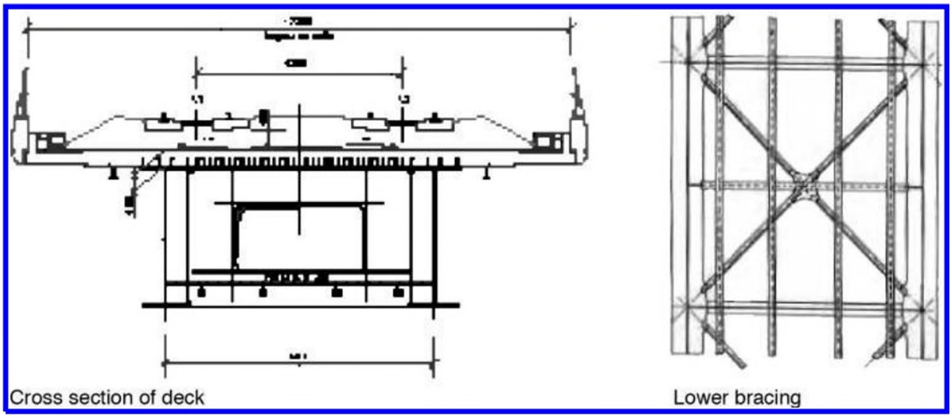

Figure 1. Composite twin girder deck.

The conceptual design was carried with special attention to the dynamic behaviour of the deck which had to be guaranteed under the high speed train. This is one of the main reason why the high speed railway steel bridges systematically incorporate a concrete slab which takes part in the global and local resistance as part of the composite deck structure. It also carries the ballast. The concrete also brings supplementary mass and damping, thus decreasing the noise emission under the TGV passing. High speed requires the steel deck to be very stiff and sufficiently heavy to limit dynamic phenomena, which have to be mastered in order to ensure the safety and comfort of the train passengers. The railway works regulations impose severe and precise criteria for these points. This also has some important consequences on the detailed design regarding the fatigue resistance.

4 THE STEEL USED IN THE TGV BRIDGES

The large use of steel in the structural parts of the TGV bridges was the outcome of some particular factors concerning the steel material:

- The conceptual design of the steel parts was optimised according to the maximum dimensions provided by the rolling mills: lengths of 36 m, widths of 5200 mm, thickness of 150 mm and weights of a single plate reaching 36 tons (Fig. 2). This allowed the use of only one plate with the flange thickness precisely tailored to the longitudinal bending moment: the maximum height used was over 5 m and the maximum thickness reached 150 mm. As for the webs, the high shear forces required up to 30 mm plates – this had the benefit of reducing the stiffening in service and launching.
- The steel grades and qualities depending on the thickness of the plates were provided without any difficulty according to the European standards: from 30 to 150 mm it was the fine grain high strength steels S355N and S355NL. For thickness below 63 mm, the thermo-mechanical S355M and S355ML grades were used. The TM rolled plates with an increased weldability without pre-heating allowed significant cost reduction in fabrication and site works.
- The use of longitudinally profiled plates (LP plates) was significantly increased. These plates are specially rolled with a precise variation of thickness longitudinally. The design engineers can thus tailor their steel requirements exactly according to the calculation results. This leads to important cost reduction at fabrication, both by decreasing the number of welded junctions and the plate thickness at these welds. The overall gain can be estimated to 10% of the steel cost. About 4000 tons of LP plates were used on this TGV line.

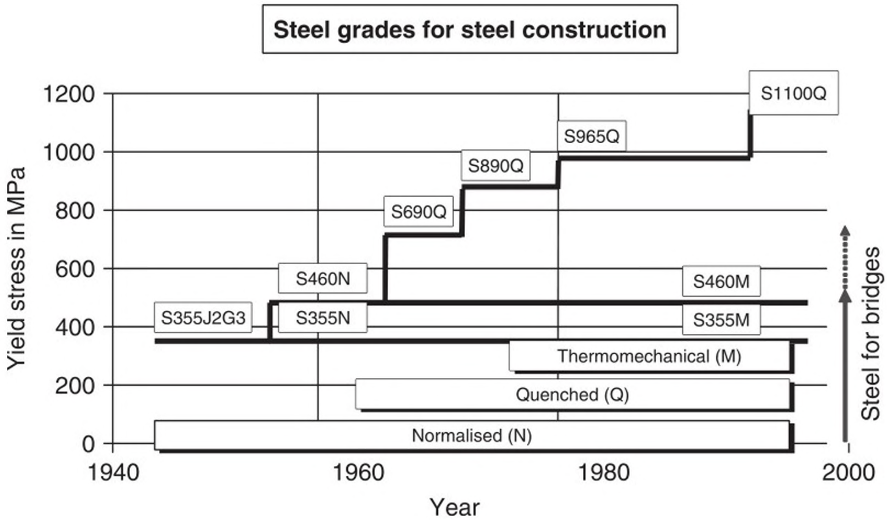

Figure 2. Steel plate grades.

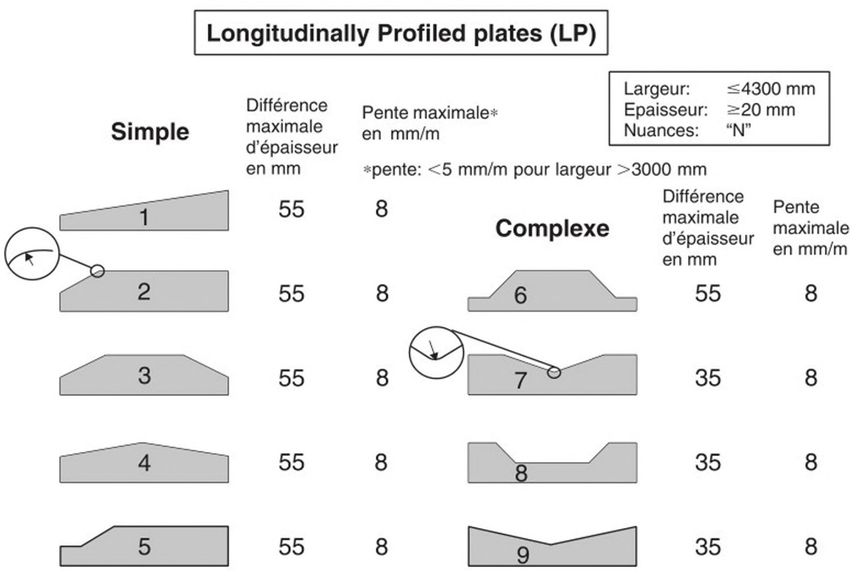

Figure 3. Longitudinally profiled plates.

The economy of LP plates in twin girder bridges is more significant in the large continuous viaducts with many equal spans; they are especially interesting in the pier regions.

– Special Z35 steel was imposed for plates subjected to tensile stresses outside their plane: the inner web of arch ribs at the junction with the first cross beam of the deck. In this plate, extra control during the steel fabrication guarantees against the risk of lamellar tearing.

5 SPECIAL POINTS CONCERNING THE STRUCTURAL DESIGN AND THE SHOP FABRICATION

In all these bridges, particular attention was given to the integration of architectural and structural values in the engineer's work through a constant dialogue with the architect [1]. This gave birth

Figure 4. Complex fabrication of tubular nodes.

to the spectacular steel superstructures in the large tied-arch viaducts with complex fabrication details.

In order to ensure safe and uninterrupted service over a period of one hundred years, fatigue analysis of all structural components have to be carried out. This was done with the usual method of fatigue coefficients based on the detail category of Eurocode 3.

The dynamic calculations that are required to ensure that comfort criteria are respected also give the stress cycles for fatigue verification of fabrication details. The careful fatigue design is achieved by limiting the number of assemblies that give rise to stress concentrations because of their shape and construction. The lower steel bracing in the twin girders are thus assembled with high friction bolts on carefully shaped gussets cut in the lower flange plate. Nodes for tubular lattice girders were carefully cut and fully welded, with a detail category of 50 MPa. In order to ensure safe and uninterrupted service over a period of one hundred years, fatigue analysis of all structural components have to be carried out (Fig. 4).

As explained above, the lower steel bracing were bolted on the flanges in the twin girders. This particular junction is the only bolted connection. The steel structure is otherwise completely welded. In the factories all the shop works are highly automated. The complex details are fabricated with computer driven cutting machines. For the plate girders, automatic welding machines are used for height up to 5 m or more. This alone largely accounts for the competitivity of the steel girder bridges.

One of the basic ideas in the design of the steel assembling was to allow the preferential passage of main elements through secondary elements at their intersection. This gave rise, in some cases to some difficult fabrication.

For all the bridges, a blank mounting was carried in the fabrication mills to ensure a correct presentation of the steel elements on site.

6 THE CONSTRUCTION METHODS

The diversity of the structural types of the TGV bridges, as shown in Table 2 was evidently reflected on the construction methods of each bridge, adapted to the particular site conditions and the specific means of the steel construction company.

6.1 *Composite girder decks*

For the composite girder bridges, the construction phases did not lead to higher forces and stresses than the service conditions because TGV design gives very rigid bridges in which the stresses attained in the mounting phases, especially the launching phases are much lower.

Figure 5. Launching nose.

Figure 6. Construction platform.

The launching method was the most commonly used method, over railway lines, motorways, rivers and valleys. In order to limit end deflections, a tapered plate girder or a truss was added in front as launching nose for most of these bridges. This nose has a length of approximately a third of the longest span (Fig. 5). Some construction company preferred to launch without a nose; but in this case the roller supports on the piers were mounted very high on steel beam stack and the support lowering was much longer at the end. In the Bonpas Viaduct over the busy A7 motorway, the steel structure was launched with the prefabricated slabs already fixed between the lower flanges.

When the four girders deck were launched the launching support was installed under the two external girders only.

Lifting by crane was often used for the girder bridges. This required, of course a special track for the mobile cranes, as for the Cavaillon Viaduct over the Durance, a complete embankment was built across the river all along the bridge (Fig. 6). In other sites, the crane was chosen with a sufficiently long reach to move the girders into position.

6.2 *Double bowstring of La Garde Adhémar (Fig. 7)*

This bridge is situated over the navigable canal along the Rhone. The important skew of the line and the respect of navigation clearance have resulted in two main spans of 115 m with approach

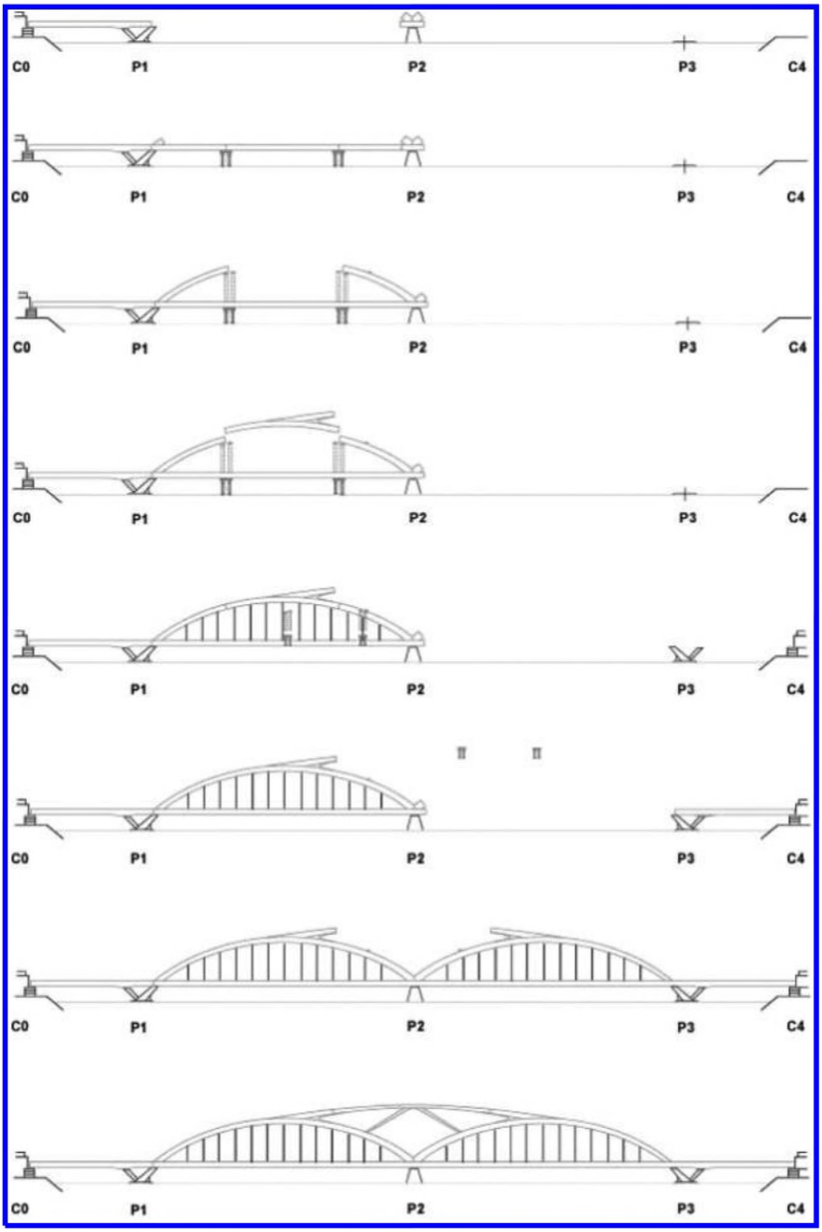

Figure 7. Erection of double tied arch on temporary pier.

spans of 47 m. It is the only bridge of the line with steel piers. These are fixed on the concrete foundation blocks built inside cofferdams in the canal. The side piers with a tetrapod form are fixed to the end spans. All the piers have box sections filled with concrete to resist boat impacts and to provide global stiffness. The arch ribs and tie beams also have box welded sections.

First the side spans were mounted with cranes and welded to the steel piers. Two temporary piers were installed successively for each span with the traffic diverted to the other half. First the deck was mounted by a pontoon crane in three parts. Temporary columns were then installed over

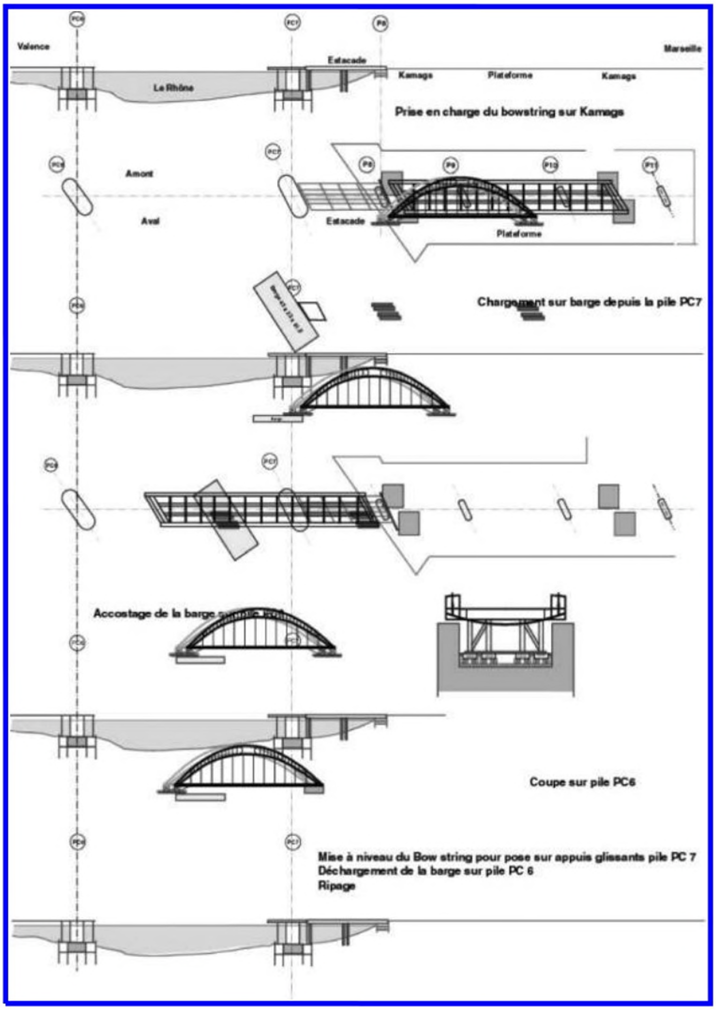

Figure 8. Erection of tied arch by river barge.

the temporary piers for the arch ribs. After the mounting of the other span, the tied arches were completed with the central tie and inclined struts. About 18 months were required for the steel works.

6.3 *Double rib Mornas and Mondragon Tied-Arch Viaducts (Fig. 8)*

To cross the Rhone meanders north of the town of Orange, the viaducts of Mornas and Mondragon each consists of a main tied-arch span respectively 84 and 121 meters long.

The access spans of composite twin beam decks have variable lengths around 50 m and different pier skews. The double ribbed arches are bounded by vertical posts over the hangers. The construction of the tied arch spans started with the assembling carried on temporary props on the access embankments. The mounting then continued by moving the completely assembled steel structure with the front end propped on a launching barge and the back end moving over the already built shorter access spans for Mondragon and a temporary rolling platform for Mornas. The longer side twin girder viaducts were launched from the embankment while the isostatic span was built by craning.

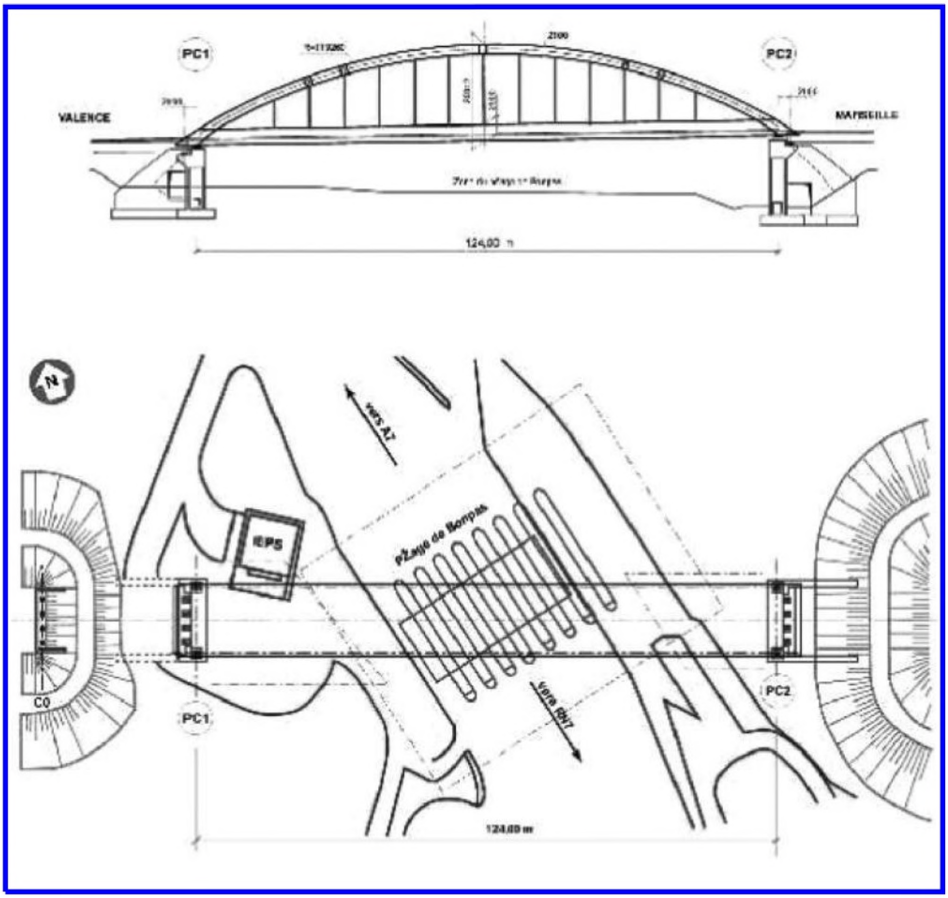

Figure 9. Tied arch carried by multi wheel carrier.

6.4 *The bowstring over the South Avignon toll station (Fig. 9)*

The tied arch over the A7 toll at the south Avignon interchange has a more classical design. The span length of 124 meters is however the longest on the high speed lines in France. Because of the toll buildings right under the bridge, there was no possibility of crane construction and temporary piers. For this reason, the bowstring was completely assembled on the north embankment and carried over the toll with the front end supported on steel props on “Kamag” rolling platform moving with numerous wheels coupled together and computer controlled.

The carefully prepared operation took place in less than four hours.

6.5 *The Arc Viaduct (Fig. 10)*

The Arc Viaduct at the south end of the line near Aix-en-Provence has nine equal spans of 44 m. With a remarkable tubular lattice structure below the composite deck this bridge shows an architectural and technical audacity seldom seen on a railway line. The choice of this particular design was also justified by the presence of the arched masonry aqueduct of Roquefavour in the background of this beautiful valley.

Figure 10. Craning of truss girder bridge.

The truss girders had been transported from the factory and were assembled with the cross girders on the platform along the piers. The lifting of the steel structure was carried in only one week with a heavy crane. The maximum weight of the elements reached 250 tons over the river span.

The lifted elements were temporarily fixed to each other, before aerial welding was completed.

6.6 *The TGV East bridges*

The new high speed railway line TGV EAST under construction between Paris and Metz has many large bridges in steel. For the complex skewed crossings, an elegant and economical solution has been the design of decks with lateral steel girders (half through bridges) and composite steel concrete deck. This was specially the case fort the A4 motorway crossings with low clearance in the Marne district:

- VIA 23B245 at Bussy-le-Château,
- VIA 23A255 at Billy-le-Grand,
- Viaduc de l'Orxois at Château Thierry.

The construction could only be carried by incremental launching over the very busy motorway as the traffic could not be interrupted (Fig. 11).

The launching method was also used for the double box composite deck of Jaulny Viaduct in the Moselle valley (Fig. 12).

7 HSR BRIDGES IN OTHER COUNTRIES

The French technology for high speed railway bridges has been exported overseas.

Figure 11. Launching of Orxois bridge over A4.

Figure 12. Launching of double box girder of Jaulny Viaduct.

7.1 *The Korean high speed rail line*

For the high speed line between Seoul and Busan in Korea, different types of composite bridges have been built, in seismic area.

Many twin I girders composite bridges with various spans were built. The following structures were designed in 1998–1999: 1@50 m, 2@50 m, 1@35 m, and 40–50–40 m. They used 335 MPa yield stress steel, and in situ welded connections. Simple spans and 2@50 m spans were built with a crane. The 3 spans bridge was launched.

For longer spans, two exceptional bridges have been designed.

The Moam #2 bridge consists in a truss span of 65 m. The two very light Warren type truss girders are located under the concrete slab. The bridge has been launched with a launching nose over a highway (Fig. 13).

Figure 13. Moam #2 composite truss bridge on Korean HSR line.

Figure 14. Moam #1 tied-arch bridge on Korean HSR line.

The Moam #1 bridge is a tied-arch with a 125 m span, which is the world record span for a tied-arch bridge supporting a HSR line. The hangers are made from 180 mm steel bars. The composite deck includes four stringers and I shape cross beams supporting a concrete slab. The arches have a rectangular section. The bracing consists in only three rectangular beams located between the arches. The bridge received an award in Korea. It was assembled parallel to a highway, and then was set in place by rotation above the highway in operation.

7.2 *CTRL in England*

The Channel Tunnel Rail Link (CTRL) is a 109 km long high speed rail line connecting the Channel Tunnel and London.

Many I girders composite bridges using the French technology have been designed for this line, each with 3 to 8 spans ranging from 21 m to 43 m. One of them is a frame structure with inclined legs. On site connections are bolted. The steel used is S 355.

Figure 15. Truss bridge on Taiwan HSR line.

7.3 *The Taïwan high speed rail line*

The Taïwan high speed rail line will link Taipei to Kaohsiung. It is located in very seismic areas.

Several twin I girders composite bridges have been built, with 50 m spans.

A composite bridge with 50–85–50 m spans used four I girders.

Three special truss bridges have been designed, with 96.50 m or 104 m spans. They consist in lateral Warren type truss beams and a composite deck, which includes four stringers and I shape cross beams. Top and lower members are rectangular boxes. Diagonal members have a H shape. On site connections are bolted. The bridges have been launched over railway lines using a launching nose.

8 CONCLUSION

In its comeback in the TGV bridges, confirmed in the construction of the spectacular Viaducts of the TGV Méditerranée, steel bridges are now recognised as the most economical type. The complex structural design of the tied-arches, tubular trusses or the plate girders with the composite steel-concrete decks have proven that steel has been able to adapt to new economical and technical necessities.

At the same time, the steel bridge constructors have also put forward challenging construction methods which have been successfully carried out without any accident.

This trend was confirmed in the TGV East line under construction now with the choice of composite steel decks for all the large bridges.

The French know-how in this field has been also applied in many other countries.

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CHAPTER 2

The effects on the interoperability of the European Railway Traffic of European Standards

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ABSTRACT: EN 1991-2 "EUROCODE 1 – Actions on structures – Part 2: Traffic loads on bridges" was published by the European standardisation organisation – Comité Européenne de Normalisation (CEN) in the English original version as a European standard in September 2003. This standard is also described as Eurocode 1 part 2 in the series of the Eurocodes.

In this standard were described the actions on bridges for road bridges, pedestrian bridges and for railway bridges. These actions do not only apply to bridges but also as load specifications for other structures, e.g. for railways for all structures charged by railway traffic.

The following article shows the backgrounds and the introduction of the European standards and describes their effects on the interoperability of the European railway traffic.

1 INTRODUCTION

The EN Eurocodes will be set of standards that contain common unified calculation methods to assess the mechanical resistance of structures or parts thereof.

They will be used:

- to design structural construction works (building and civil engineering works)
- to check their conformity with Essential Requirement no 1 – mechanical resistance, including aspects of Essential Requirement no 4 – safety in use, and partly Essential Requirement no 2 – safety in case of fire, including durability, as defined in Annex 1 of the Construction Product Directive
- to determine the performance of structural construction products (Construction Product Directive).

The EN Eurocodes will be organised in currently 55 parts covering Actions, Steel, Concrete, Composite Steel and Concrete, Timber, Masonry and Aluminium, together with Geotechnical design and Seismic design. Each part will be published as European Standard.

The Eurocodes will become the Europe wide means of designing Civil and Structural engineering works and so, they are of vital importance to both the design and Construction sectors of the Civil and Building industries.

2 HISTORY OF THE EUROCODES

The elaboration of the Eurocodes has started in 1974, on the basis of an initiative taken by the Profession (group of industrials and research association). From 1975, the European Commission has supported this initiative. Between 1984 and 1988, a first series of documents were published.

Then, the elaboration of the Eurocodes was transferred to CEN (Comité Européen de Normalisation) in 1989, to be elaborated in a format of European standards.

In 1992 the European Commission engaged CEN to elaborate the EN Eurocodes. Then the Technical board (BT) decides of the following resolution:

It supported the resolution of the BTS1 and asked to establish this relation for the Eurocodes to all working groups.

Then in the resolution BTS1 11/1992 the bases for the corresponding work of CEN/TC250 were fixed.

Resolution BTS1 11/1992.

In this resolution the responsibility for the Eurocodes and the assigned dependencies are defined, within the same Technical Committee and outside at other committees and standardisation bodies.

This resolution has considerable consequences, while it is also the bases of the internal co-operation and the individual combinations between the design and product standards.

The Technical Committee 250 (TC 250) was established in 1990 after this order and developed the first Eurocodes as "pre-standards". At present, this committee is transforming the preliminary standards into European Standards.

A first series of 62 pre-standards – European preliminary standards (ENV) – (actually, the second generation of "Eurocodes") was published between 1992 and 1998. These ENV's contained many choices (safety coefficients, values and alternative methods) so called "boxed values", which can be chosen by the planner or be fixed at the national level. Therefore the parts of the ENV Eurocode were completed by "National Application Documents" (NAD), published at the national level, which indicate how the Eurocodes could be used in each Member State.

The present structure of the TC 250 with the necessary working groups for the construction of the Eurocodes is shown in Figure 1. So all parts 1–2 are responsible for the fire protection, all parts 2 are for bridges. These parts are controlled respectively by a Horizontal group which also takes on the corresponding co-ordination and control. Normally not all Eurocodes have these two parts.

3 ELABORATION OF EUROCODE 1 PART 2

In the field of the European railway operators already existed for years regulations by the International Railway Union (UIC) which forms a common base for the railway infrastructure of all UIC members with partly obligatory and partial recommending regulations.

While all European member nations of the UIC are also represented in the EU, it seemed reasonable to use the UIC regulations as a basis for the railway specific regulations in the European standards.

Furthermore, there existed and still exists no general standards by the previous official status of most railway organisations for the railway infrastructure. Here were the organisation-related regulations valid, which have also normative character.

Due to these backgrounds the working group for the elaboration of the Eurocodes in the railway sector was formed by UIC members and they used the leaflets of the UIC as a basis of the new European regulations (see Fig. 2).

This working group elaborated the special railway specific actions for railway structures as part of the Eurocode.

The bases was the load model of the UIC, the load model UIC 71, which was developed in 1971 and then gradually introduced by the different railway administrations.

The expression "load model 71" (LM 71) was chosen for the Eurocode to avoid association with specific names.

The same working group then organised the revision of ENV 1991-3 to EN 1991-2 in the years 1997–2001. This working group (PT2) was managed by Mr Calgaro (France) for the road sector and Mr Tschumi (Switzerland) for the railway sector. Mr Tschumi was chairman of the UIC panel of structural experts at that time.

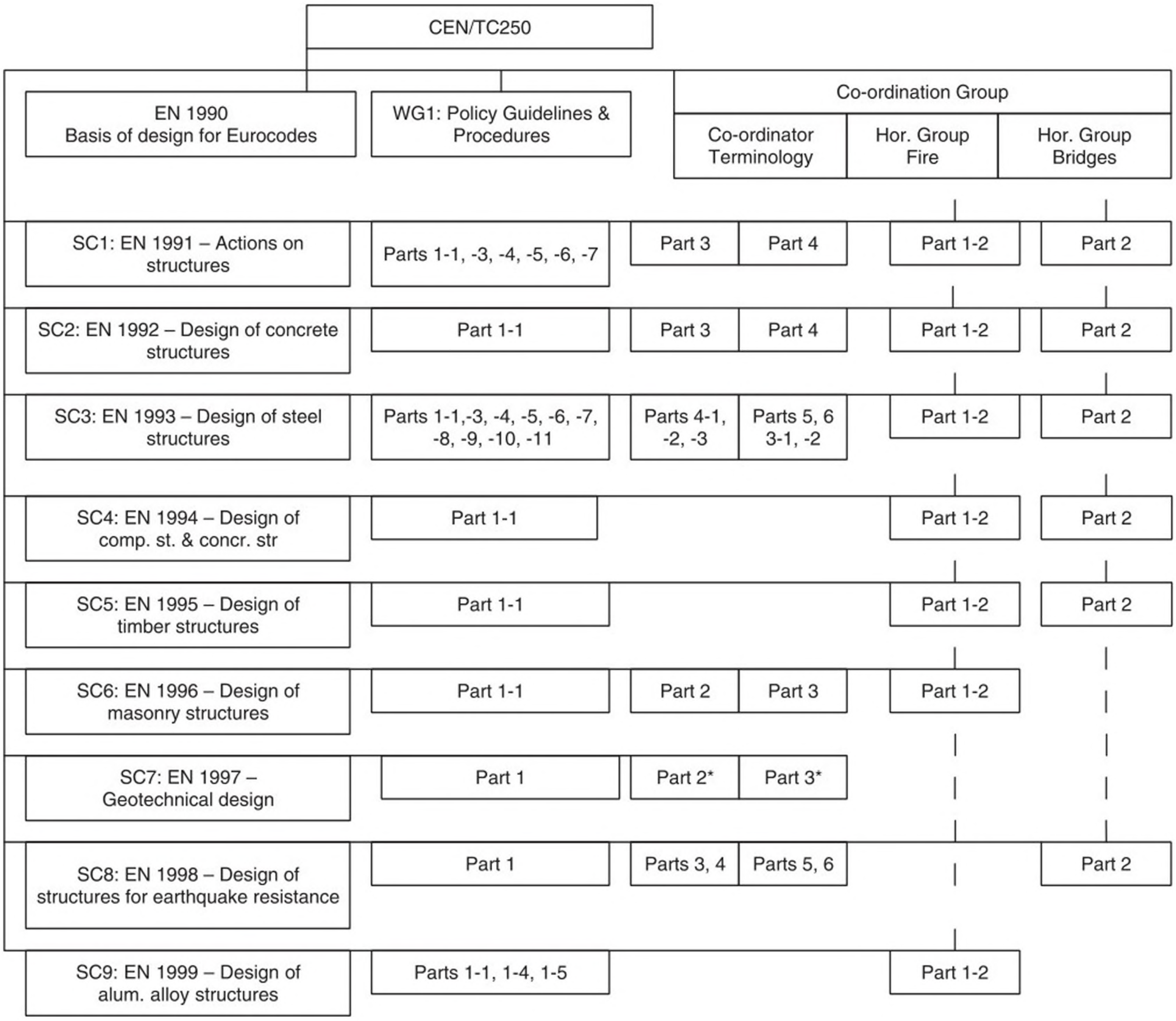

Figure 1. Representation of the structure of the Eurocodes [1].

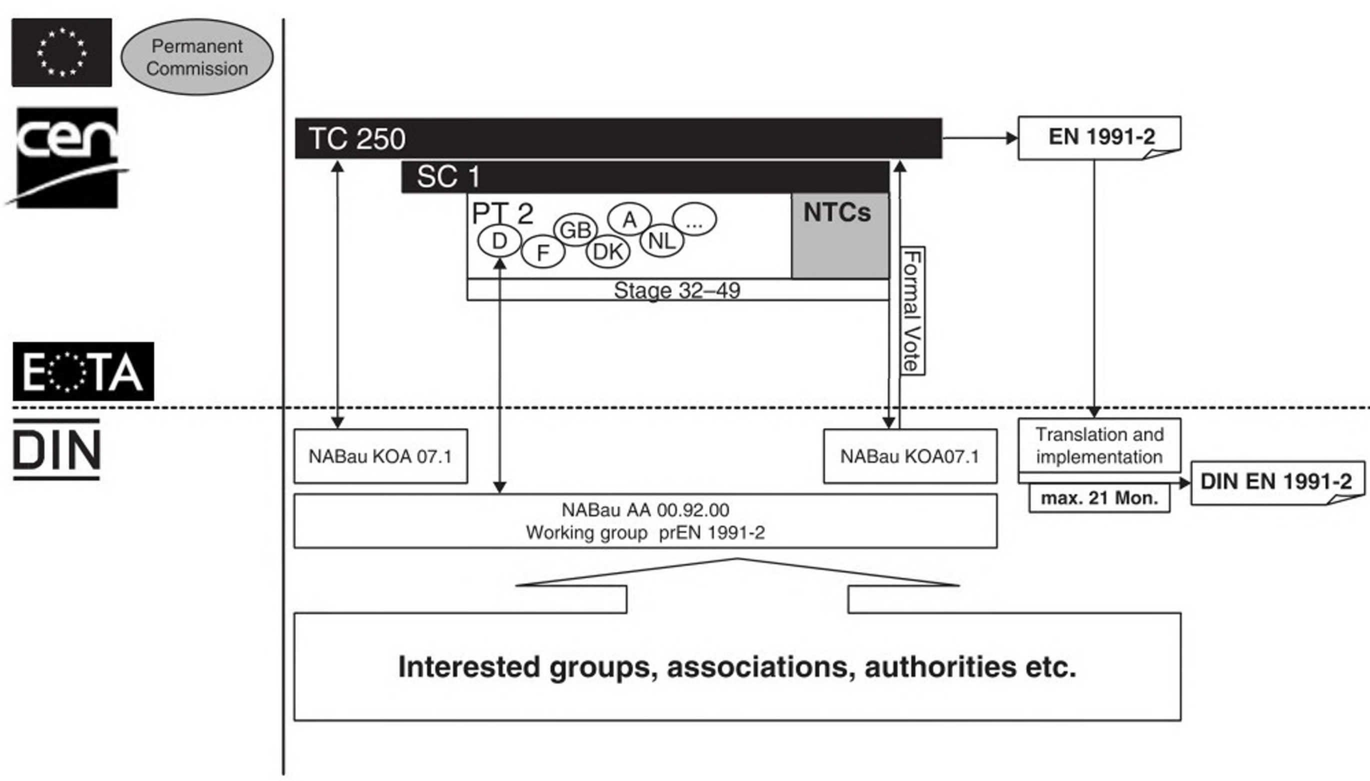

Figure 2. Representation in Eurocode working groups.

In different meetings which partly took place as working meetings with technical subgroups but also as complete meetings with almost 50 participants, the changes for the ENV of the individual countries and organisations were discussed and included. Between the meetings the different drafts were distributed to the national contact persons (NTC – National Technical Contact) who for their part involved further experts at national level and elaborated the official national comments.

The draft completed in the working group was transferred to the EU Member States for Formal Vote, the official Vote, and was accepted with only one vote against.

Subsequently the language design of the standard draft was then improved and additional explaining notes were written without any change of the regulation by a small editorial staff team. In this respect, it was of great importance while the three language versions (English, French and German) have to stay besides each other similarly and do not allow any different interpretations. This work was completed in May 2003.

The CEN management centre then got the final document, which was published in September 2003. This date indicates the so-called date of availability (DAV), on which the standard was available for the public. A procedure follows which indicates the further processing in the EU and ends with the withdrawal of national standards in this field – the date of withdrawal (DoW).

EN 1991-2 is part of the Eurocodes as said already and is a design standard in this respect. It provides the basis for a set of other standards, which regulate products, however, is base also by its general character for other design standards which deals with bridges or with general railway actions.

In Figure 1 the combinations with EN 1992-2, 1993-2, 1994-2 1995-2 and also 1998-2 are shown. As an additional basic standard EN 1990 must be taken into account. The bridge specific parts are combined in the appendix A2 of EN 1990 which was submitted to Formal Vote at the end of 2003.

From these connections and the planning to 2010 can be recognised very fast that a direct temporal transfer into a national set of rules is only possible if the allocated standards also exist and are applicable.

So-called packages to the Eurocodes were formed which includes several standards to facilitate the applicability. These packages could be found among others in the guidance paper L, appendix C of CEN/TC 250.

By the different safety philosophies of the national and European standardisation it isn't ensured that both series of standards can be used with each other. There is a strict prohibition to mix standards with different philosophies.

The European Commission has asked all Member States by publishing of its recommendation of 2003-12-11 to use the EN Eurocodes immediately after their publishing.

The TCs listed in table 1, which only prepared and published a few standards, are allocated among others to EN 1991-2.

Surely, it is not practicable to introduce a design standard with specifications which cannot be fulfilled by the industry. At the moment, it concerns among others prestressing steel whose licensing must be adapted to the new requirements.

There are relatively few changes for the railway organisations in Europe in the design requirements as mentioned, while the UIC leaflets were basis for the railway specific requirements.

However, the product standards will certainly have consequences in all sectors of construction.

If all relevant European standards were elaborated and published till 2008, they will be introduced at the same time and are then substantial for the building activities in Europe.

Until this date the new European standards can be used voluntarily in the general business, they are not compulsory for normal users.

The application of standards for construction is regulated nationally which produces certain differences between the different countries. There is still more freedom for the private client than in the public area.

However, among the railways other regulations are still important now at European level which, at long last, stipulates a mandatory use of European Standards for civil engineering structures with all the problems which were already mentioned. Whether this happens by the respective companies or the relevant authority now depends on the national structure of each railway organisation.

Table 1. Allocated TCs [1].

CEN/TCs	Title	Related Eurocode
33	Doors, windows, shutters and building hardware	1
50	Lighting columns and spigots	1, 2, 3
53	Scaffolds, falsework and mobile access towers	3
104	Concrete	2
112	Wood based panels	1, 5
124	Structural timber	5
125	Masonry	6
129	Glass in building	1
167	Structural bearings	3, 8
177	Prefabricated reinforced components of autoclaved aerated concrete and lightweight concrete with open structure	2
226	Road equipment	1, 2, 3
229	Precast concrete products	2
242	Passenger transportation by rope	1, 2, 3
284	Greenhouses	1, 9

4 INTRODUCTION OF THE EUROPEAN STANDARDISATION IN GERMANY [2][3]

As an example the introduction of the European standardisation in an EU state shall be represented by Germany. Here, the already existing ENVs were introduced as relevant standards with additional National Application Documents (NAD), which shall be integrated in the EN as a national appendix (NA). The redress on the ENV permits an immediate applicability, however, it holds the uncertainty of another change when introducing the EN.

The ENV were published together with the NAD and broader accompanying standards as a "woven document". These are the DIN technical reports 101–104.

In Germany, a corresponding applicability for many areas was achieved in addition by exceptional approvals or customisation regulations.

Subsequently the introduction of these DIN technical reports at the Deutsche Bahn AG is shown and the thus coherent sets of rules of the Deutsche Bahn AG and the BMVBS (German Federal Ministry of Transport, Building and Urban Affaires).

The introduction of the DIN technical reports as a transfer of the European standards was carried out on May 1st, 2003 as pointed here (see Fig. 3).

Due to the contractual agreements of the Federal Republic of Germany with the German institute for standardisation also other legislative actions are then connected.

Parallel to the standards the entrepreneurial requirements of the Deutsche Bahn AG had to be adapted for the new situation.

Furthermore within the structures of the European standardisation it still has to be taken into account that there are two kinds of standards with a different legal status. The design standards are standards which are in principle simple and not harmonised at which different values and methods can be provided within a design regulation by the national annexes.

The product standards are harmonised standards which are mandated by the European Commission and at which national differences are allowed only to choice limits or classes and methods in a range of before established methods and values.

The reference to another standard means that this regulation gets the higher-order status automatically.

As earlier mentioned, European standards don't have an influence only by their own form. Also other regulations make use of it and bring it therefore directly in the domain of railways.

The introduction of the European standards in the railway sector will be carried out in three ways:

1. As already described as an independent EN. The validity of the EN confines itself directly to the CEN Member States. An EN has the legal character as "state of the art".

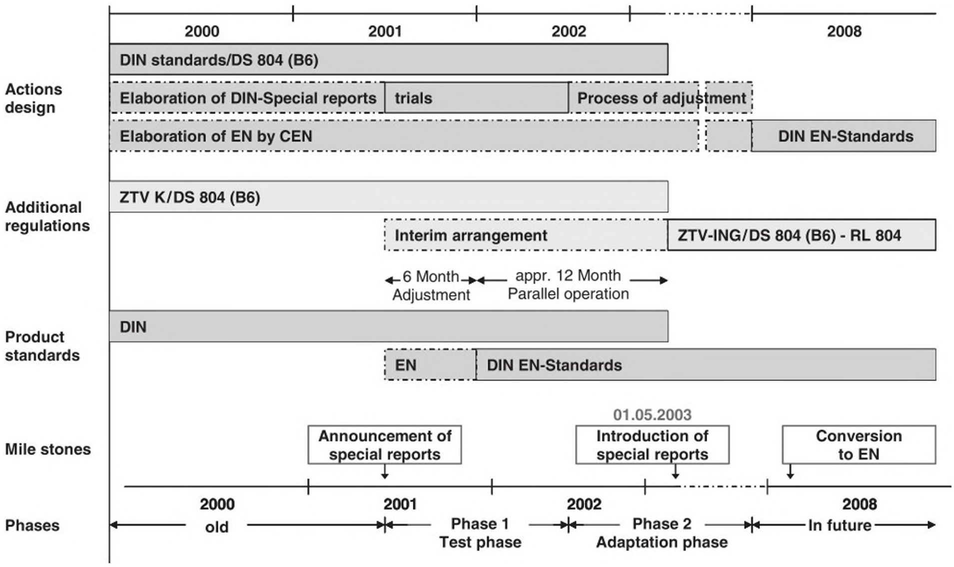

Figure 3. Introduction of the EN standards in Germany.

2. In the context of the technical specifications for the interoperability (TSI) as an execution instruction for the EU Directive 96/48/EC. Due to this the quoted parts have legal character for the EU Member States.
3. In the context of a UIC leaflet. Meanwhile, the UIC leaflets only have in Europe the rank of informative documents, if they aren't quoted by other regulations correspondingly and thereby get the rank of this regulation (e.g. by EU Directives and others).

At the moment 57 of the 58 parts of the eurocodes are published in Germany as German standards DIN EN and will be used in future for the design and construction of structures and buildings in Germany.

5 INTRODUCTION IN THE CONTEXT OF TECHNICAL SPECIFICATIONS FOR INTEROPERABILITY (TSI)

In accordance with the EC Directive 96/48/EC of 1996 the technical specifications were elaborated for the interoperability of the high speed rail traffic. These TSIs were introduced on Nov 30th, 2002 and are obligatory in the Member States of the EU.

Among other things the previous ENV 1991-3 is referred in the TSI infrastructure to whose normative regulations get a legal character with this reference. At the present revision of the TSI infrastructure this normative reference will be adapted to EN 1991-2 to establish the current reference.

In the TSI sections 4.3.3.3 and 4.3.3.13 to 4.3.3.15 the requirements for civil engineering structures are described and referred to the corresponding requirements of Eurocode 1.

This is the same in further TSIs.

The TSIs are obligatory in the Member States of the European Union as mentioned. At present, their regulations are referring only to the high speed lines which were coming into service since Dec 1st, 2002. In the next step the TSIs were elaborated for the conventional lines which then will be substantial for the lines of the TEN network, but infrastructure doesn't have any prior mandate (Table 2).

With the beginning of the year 2006 the new European Railway Agency ("ERA") took over the work of the AEIF and will work on the TSIs in the future. In May 2006 they got the new mandate

Table 2. Mandates for TSI, [4].

	High speed revision	Conventional rail
Control-command and signalling	X	X
Operations	X	X
Rolling stock	X	freight wagons only
Energy	X	interfaces only
Infrastructure	X	interfaces only
Maintenance	to be included in the relevant TSIs	–
Accessibility people with reduced mobility	–	X
Safety in tunnels	–	X
Telematic applications	–	freight only

Table 3. Mandates for TSI for ERA.

	Conventional rail
Infrastructure	X
Passenger carriages	X
Locomotives and traction units	X
Energy	X
Telematic applications for passengers	X

Table 4. Overview.

	(h)EN	TSI	UIC leaflet
Scope	CEN Member States	European Union	UIC member-organisations
Status	(Harmonised) Recognised rule of technology	Statutory order	Recommendation
Coming into force	CEN rules	Publication in the Official European Bulletin	Publication as UIC codex

for the TSIs (Table 3). Besides these TSIs the ERA will also develop the common safety targets ("CSI") and the common safety methods ("CSM") relating to the railway system.

These TSIs have to be finished within 3 years after begin of working.

By these references a legal character is created as mentioned which on one hand increases the value of the standard but reveals on the other hand the already discussed difficulties. If the allocated product standards still aren't available, there exists a legal obligation which isn't soluble.

This indicates generally the great dilemma in which is the general construction business in the European Union at the moment. By radical change in standards and regulations there has been a certain uncertainty at clients, planners, managers and for the relevant authorities for years.

Therefore all involved persons are asked in this field to approach the solution of the problems with a common sense and pragmatically. A too formal procedure generates more problems than benefit. This shall not indicate to neglect the necessary safety procedures or to analyse decided regulations. All involved persons must rather try to get together a solution for the upcoming problems.

6 INTRODUCTION IN THE CONTEXT OF UIC LEAFLETS

The UIC leaflets formed the basis of the standardisation in the field of the railway specific civil engineering structures within the last decades.

So the regulations from the UIC leaflets 700 R, 702 OR, 776-1 R, 776-2 R, 776-3 R, 778-1 R, 779-1 R and others as well as from the reports of the ERRI expert groups were taken into the Eurocode. While a further development was connected to the elaboration at the same time of course, the current regulations are taken over in the current UIC leaflets, sometimes with additional explanations to distribute the current knowledge to the complete railway comunity.

As seen at the above enumeration, the UIC leaflets have mostly a status of a recommendation (R). They are put at a disadvantage in the context of the other sets of rules in Europe because they only have the lowest legal status. On the other hand the UIC leaflets can be changed liberally to the wishes and requirements of the railways without passing through a formal and long-term legislation method.

7 SUMMARY

Due to the different deregulation methods in Europe the process of the restructuring of the different railway organisations in the Member States of the European Union isn't completed yet and since the system railway is a world-wide system, both the aspects of the European railways and the interests of the other railway organisations existing in the world must be taken into account at all changes and further developments. This is a great challenge for everyone who is involved in the elaboration of standards.

It is not only the regulating of single requirements but rather the analysis of individual measures in the frame of the complete system. A standardisation committee like CEN looks at the regulation object purely with a regulating view, here with a technical view with a finding of the current state of technology under consideration of science.

The EU commission prefers the competition and describes the individual components of the system railway in the TSIs but with well defined interfaces to the respectively other components to ensure the totality of the system.

For the UIC is on one hand the world-wide connection of the system railway in the foreground, furthermore but also the further development and support of the available systems. The research is initiated and a control from the view of the operators is brought in here.

The Eurocodes gives a common base of the calculation methods for the European building and construction industry to create similar cross-border utilisation possibilities. For the railways which were always run cross-border, it is only another logical consequence in the common bases and requirements for their system.

The standards must be used now so that the system railway can further improve its advantages on an international level on the one hand and on the other hand can serve as an example for the other construction industry.

Additional information can be found on the following websites:
www.cenorm.be
www.aeif.org
www.era.europa.eu
www.uic.asso.fr
www.eurocode-online.de

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CHAPTER 3

Railway bridges for high speed lines and Eurocodes

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ABSTRACT: High speed railway bridges can now be designed using the new European standards "Eurocodes for construction". Many rules and recommendations concerning Service Limit States, actions and calculations are given in two main parts of these Eurocodes: – Eurocode EN 1990 Annex A2 and EN 1991 – Part 2 section 6 «railway bridges». These two standards resume research works based upon the experience in the high speed field of different European railway companies (among them SNCF and DB) unified in the UIC organisation. These works have been put together and published into UIC codes (UIC leaflets). This lecture first recalls the Eurocodes rules in the dynamics field and then gives some background information concerning those rules: – Why static calculations are no more sufficient to model the effects of a train running across a railway bridge?; – how to decide whether dynamic analysis is required?; – how to model the bridge and trains running on tracks over bridges?; – what are the criteria for a good behaviour of the structures (vehicles, track and bridges). This background information is mainly based upon SNCF experience. Some other dynamic model problems are presented and, in conclusions, future research topics are identified.

1 INTRODUCTION

One of the most important point to analyse when designing high speed railway bridges is the dynamical behaviour of the deck. The classical way to take into account dynamic effects in bridge design was the use of a dynamic factor (based upon few dynamical parameters) and to multiply all static effects (deflection, moments, stresses...) with this dynamic factor. Unfortunately, the French experience, confirmed by dynamic analyses, showed that this method doesn't cover some resonance effects among which those due to the excitation caused by high speed long trains with regularly spaced bogies. One of these effects (vertical acceleration) may cause destabilisation of the ballast and instability of the long welded rails. Thus it is an important safety problem.

The new European standards take into account the fact that for high speed lines the classical method is no more useable. In EN1991-2, a NOTE in clause 6.4.5 (2) says clearly: "Quasi static methods which use static load effects multiplied by the dynamic factor defined in 6.4.5 are unable to predict resonance effects from high speed trains. Dynamic analysis techniques, which take into account the time dependant nature of the loading from High Sped Load Model (HSLM) and Real Trains (e.g; by solving equations of motion) are required for predicting dynamic effects at resonance".But all this standards try to limit the cases where dynamic analysis is required (paragraph 2).

These standards also give guidance for choosing and building a satisfactory bridge-train interaction model for this analysis (paragraph 3) and all criteria for a correct dynamical behaviour (paragraph 4).

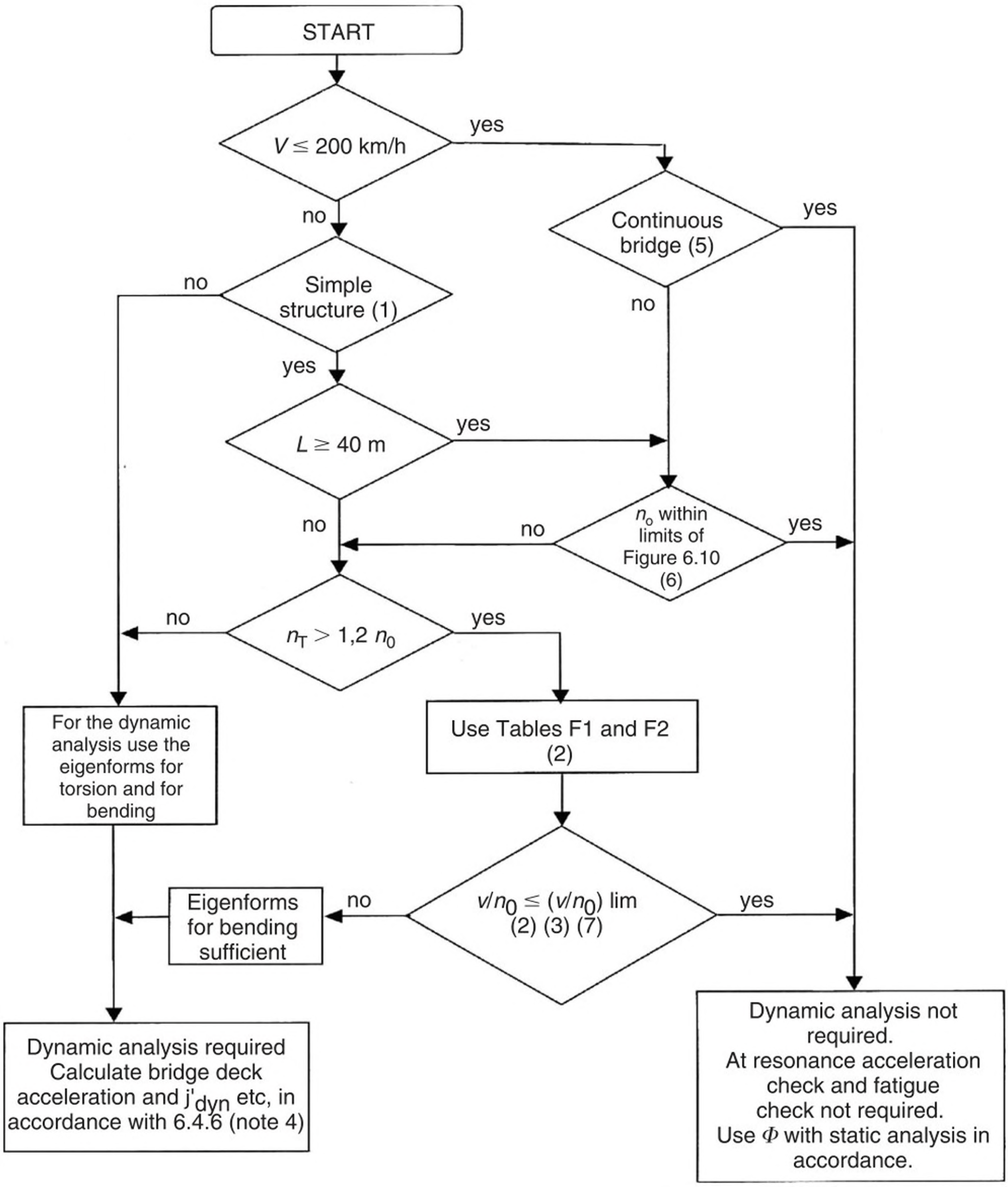

Figure 1.

2 IS DYNAMIC ANALYSIS ALWAYS NECESSARY?

The EN1991-2 standard gives a flow chart (6.4.4) allowing to decide whether a dynamic analysis is necessary or not (see Fig. 1, above).

The parameters of this chart are the following:

- maximum speed of the line;
- "simplicity" of the structure: simply supported bridges with only longitudinal line beam or simple plate behaviour with negligible skew effects on rigid supports are "simple";
- span;

– first natural bending frequency n_0;
– first natural torsional frequency n_T.

For high speed lines, for "simple" structures, where $L > 40$ m and n_0 within limits of Figure 2, no dynamic analysis is required. Limits of Figure 2 define the field where the dynamic factor Φ is valid. It is also the case for small bridges ($L < 40$ m) where the coupling of bending and torsion is not likely and where $v_{lim}/ n_0 < (v / n_0)_{lim}$, given in appendices F1 and F2 of EN1991-2. In these cases dynamic analysis has already been performed and the results are given in the appendices F1 an F2.

So, the use of this chart is not so easy and the cases where a dynamic analysis is required are numerous:

– "non simple" structures such as bridges with continuous decks, skew decks, concrete frames, concrete portals;
– "simple" structures where $L > 40$ m and where n_0 is not within the limits given in Figure 2;
– "simple" structures where $L < 40$ m and where $n_T < 1{,}2\ n_0$.

In France, for the East High Speed Line ("LGV Est") under construction, almost all structures have been checked with a dynamic analysis, according to this flow chart. However, our experience showed that it was not necessary to check all bridges but only one by type of bridge even if the skew or the span is different. These calculations have been done because the owner wanted to let the designers free for designing slender structures.

Another important issue is that for small reinforced concrete bridges, models for dynamic analysis are not described in EN standards and the stiffness for concrete in bending or in tension calculated with EN1992 or French "BAEL" (French rules for the calculation of reinforced concrete structures), is under-estimated. Static measured deflections are always smaller than deflections calculated with these rules. Four main reasons explain these differences:

– the influence of the track rigidity in bending for small structures become more and more important with the decrease of the span length;
– cracking in concrete is estimated with UIC loads model that are very high loads;
– the ends conditions of the structure are often different from those taken into account in models (small decks never have free bending conditions at their ends);
– the dynamic concrete modulus is greater than the static one and the real static one may be greater than the modulus considered.

3 WHICH DYNAMIC MODELS?

The cases where analytic methods can be used to solve PDE (Partial Derive Equations) representing the effects of trains running over a beam are limited to simply supported beams and some approximations have to be made even if computer programs are used.

Now, many FEM computer programs allow the correct calculation of required dynamic parameters when a train runs over any type of bridge.

In both cases, train loads, geometrical and mechanical characteristics of the bridge and track influence have to be defined by the engineer. EN standards give a precise guidance to find the input values for these calculations although no recommendations are made about the bridge models that have to be used.

3.1 *Train and load models*

Several train models are given. EN 1991-2 proposes implicitly that all these models are composed of running vertical point forces, representing maximum axle loads, applied to the bridge on the axis of the track. With such a representation, distribution of the wheel load by rail and ballast

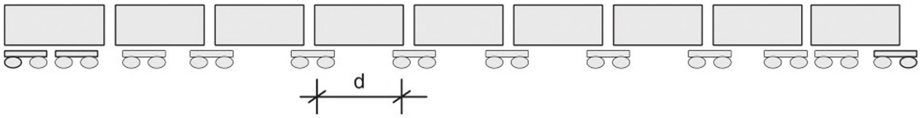

Figure 2.

Table 1.

	Nombre de voitures intermédiaires	Longueur des voitures (d)	Distance entre axe des essieux	Charge à l'essieu
A1	18	18	2	170
A2	17	19	3, 5	170
A3	16	20	2	180
A4	15	21	3	190
A5	14	22	2	170
A6	13	23	2	180
A7	13	24	2	190
A8	12	25	2, 5	190
A9	11	26	2	210
A10	11	27	2	210
Unités		(m)	(m)	(kN)

is neglected, and all horizontal forces (not taken into account are transverse centrifugal forces or longitudinal) applied to the rail are also neglected. This can be allowed for long span bridges but it is no more acceptable for small bridges.

More, the random variation of wheel forces (horizontal and vertical) exerted upon the rail and due to the fact that vehicles don't have always their maximum loads, due to movements of vehicle and to track defects may not be taken into account (see 3.3).

However, in some cases, it is necessary to model vehicles and tracks defects in order to get a better representation of forces applied to rails and to the bridge. It is the case where the geometrical quality of tracks cannot be excellent or when comfort problems have to be studied.

For interoperable high speed lines, bridges have to be designed with a high speed load model (HSLMA and HSLMB) that have been defined to cover maximum vertical accelerations produced by all European existing high speed trains (TGV, ICE, Talgo…) but also by future high speed trains that will be designed as required in annex E of EN1991-2. (see Fig. 2 and Table 1).

The advantage of HSLM is to limit the number of calculations to be performed although there are 10 models of HSLMA. Vertical accelerations calculated with this load model are always greater than accelerations due to real or future trains but the margin between the effects of each real trains and the effects of this HSLM is not known. More, HSLM cannot be used to determine effects like stresses or bending moments.

Concerning speeds, dynamic analysis is required for all speeds greater than 40 m/s as far as the maximum design speed that is 1, 2 time the nominal speed of the line. Resonant speeds have to be specially checked. In practice, calculations have to be done step by step with steps of 5 or 10 km/h. It is recommended to make a dynamic analysis with a very small speed (like 10 km/h) to obtain a reference quasi-static response.

3.2 *Bridge model*

Small unidimensional analytic models or great 3D FEM ones need inputs like geometry of the structure, mass, stiffness, damping, and boundary conditions. The way to build an efficient model is

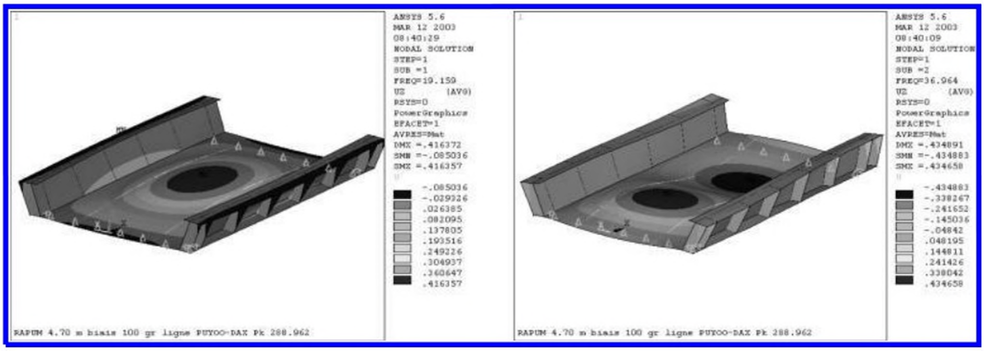

Figure 3.

Table 2. Values of damping to be assumed for design purposes.

Bridge type	ζ Lower limit of percentage of critical damping [%]	
	Span $L < 20\,\text{m}$	Span $L \geq 20\,\text{m}$
Steel and composite	$\zeta = 0{,}5 + 0{,}125\,(20 - L)$	$\zeta = 0{,}5$
Prestressed concrete	$\zeta = 1{,}0 + 0{,}07\,(20 - L)$	$\zeta = 1{,}0$
Filler beam and reinforced concrete	$\zeta = 1{,}5 + 0{,}07\,(20 - L)$	$\zeta = 1{,}5$

Note: Alternative safe lower bound values may be used subject to the agreement of the relevant authority specified in the National Annex.

given in many technical papers or books. Modal analysis is generally required for quick calculations of the response and allows to detect easily the risks of resonance. An example of mode deformation of a small steel structure is given below (Fig. 3).

3.2.1 *Mass*

The mass of the bridge is not constant all along the bridge life. Its distribution along the bridge can also vary. So, EN1991-2 indicates that two values (maximum and minimum) have to be taken into account, the maximum value corresponding to the end of a track maintenance cycle for ballasted tracks.

3.2.2 *Stiffness*

Two values have also to be taken into account. So, dynamic analysis has to be performed with maximum mass values and minimum stiffness value, and, minimum mass and maximum stiffness. In fact, at the beginning of bridge life, generally, stiffness is high and mass minimum and, at the end of bridge life, it is the opposite situation. So, natural frequencies decrease during time. Thus, if the speed of the trains is constant, it can happen, at a precise time during bridge life, that resonant effects occur when the frequency of excitation equals a natural vibration frequency.

3.2.3 *Damping*

EN1991-2 gives values for the structural damping of different types of bridges (see Table 2 and Fig. 4).

This additional damping can be used in order to take into account the effects of non suspended masses (like axles) moving on the bridge.

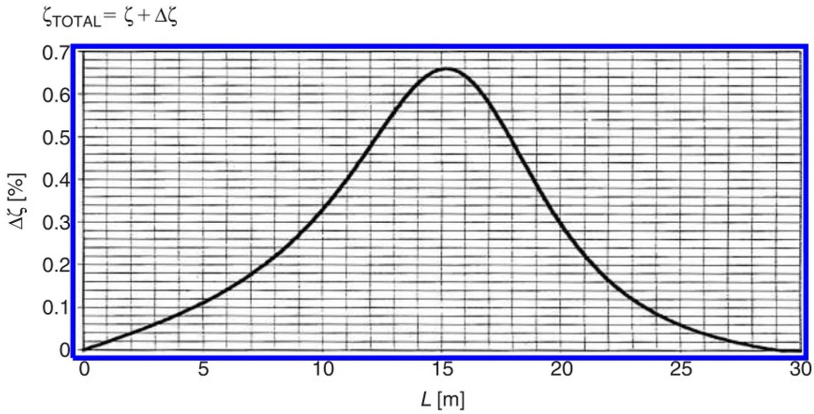

Figure 4.

Table 3. Indicative levels of comfort.

Level of comfort	Vertical acceleration b_v (m/s^2)
Very good	1,0
Good	1,3
Acceptable	2,0

3.3 *Track model*

Tracks defects can be taken into account with dynamic factors Φ or ϕ given in annex D. But these dynamic factors cannot be used for dynamic analysis.

Hypothesis about track influence are given in section 6.4.6.4 (5) and (6) of EN1991-2. It is generally considered that track on high speed line have very high geometrical quality. However, it is proposed to multiply all dynamic effects (deflection, accelerations, stresses) with $(1+\phi'')$ or $(1+0.5\phi'')$. Our experience and our practice are different. So the French national annex will propose to take $\phi''=0$ for the determination of vertical accelerations.

For complete interaction (vehicle, track and bridge) dynamic analysis, it is necessary to define geometrical track defects. One method used in France consist of using measured track defect spectra. Special dynamic calculations can take into account the random aspect of this input.

4 SLS CRITERIA

All deformability and vibration criteria are given in Annex A2 of EN1990. The most important criterion to check (where dynamic analysis is required) is the vertical acceleration of the bridge under tracks. For ballasted tracks, the peak value of vertical acceleration should not be greater than 3,5 m/s^2 "considering frequencies (including consideration of associated mode shapes) up to the greater of :

– 30 Hz
– 1,5 times the frequency of the first mode of vibration of the element being considered including at least the three first modes.

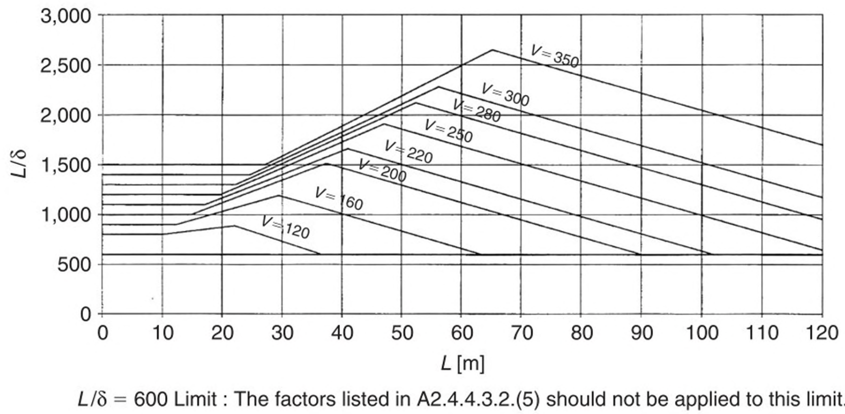

Figure 5. Maximum permissible vertical deflection δ for railway bridges with 3 or more successive simply supported spans corresponding to a permissible vertical acceleration of $b_v = 1\ \mathrm{m/s^2}$ in a coach for speed V [km/h].

Thus, it is necessary to add a filter to the whole process of calculation of the vertical acceleration. Filtering can be done according to different methods

- modal analysis and suppression of all modes with frequencies greater than the upper limit given above (but a sufficient mass has to be kept)
- numerical filtering using the time step of integration
- application of a filter after complete calculation.

These methods are to be used carefully because they may give different results.

Comfort criteria can be checked without dynamic analysis.

Static deflection has to be calculated with UIC load model and amplified by the impact factor Φ and has to be smaller than the value given in the chart below (Fig. 5).

But for very long bridges, or for "exceptional" structures or when the above rules given in A2 to EN1990 lead to too expensive bridges, a complete analysis has to be achieved. Paragraph A2.4.4.3.3 gives requirements for checking comfort for a dynamic vehicle-bridge interaction analysis. This analysis can be a coupled one or an uncoupled one. When uncoupled, the vertical wheel load calculated with the vehicle model can be different (but not very different) from the wheel load used to calculate the bridge response. When coupled, wheel load has to remain the same in the bridge calculation or in the vehicle dynamic analysis. Special computer programs allow to make these calculations.

5 CONCLUSIONS

In most cases, it is not difficult with EN standards to check the good behaviour of railway bridges for high speed lines.

However, the dynamic behaviour of small bridges, for example, is not very well known. If EN standards allow taking into account a best damping for these small bridges, it is not sufficient because all other model parameters and inputs are uncertain. Some UIC or SNCF research programs are in discussion about this item. It is now quite clear that it would be necessary to make complete rail – structures models to try to obtain a good consistency between measurements and calculations. But the link between track and bridge is not an elastic and linear one. Thus the dynamic analysis leads to use direct integration methods instead of the classical modal analysis.

Another important research theme is the dynamic behaviour of structures in interaction with soil. Bridge engineers have to learn from their colleagues who manage seismic problems.

Then comfort questions are not completely solved for very long bridges or where special vehicles would run over bridges. If this criterion would lead to too expensive structures, complete rail – vehicle – bridge interaction dynamic analysis is required.

HSLM can only be used for vertical accelerations checks but not (yet) for justifying the bridge resistance. Some researches or parametric studies should be done also for this new load model.

Another problem not completely clear at this moment is the determination of fatigue damage in steel or composite bridges when resonance occurs or when using the new dynamic load models. SNCF has some experience in that field to be completed with the new EN1993 −1.9 (Fatigue for steel construction) rules.

And now two final interrogative remarks:

– The situations where trains stops on a bridge are very rare. The static case (speed = 0) is practically never met and all our previous calculations were static calculations. When can we make systematic dynamic analysis and forget all static load models?
– Some other remaining problems (what happens with higher speeds, and so, as seen before), have to be solved. But isn't it quite normal to let some research works to our successors…?

ACKNOWLEDGEMENTS

This paper is based upon SNCF experience but also upon the works of the UIC experts committee whose head is Herr Muncke (Deusch Bahn) and the French representative Philippe Ramondenc (SNCF). Many thanks to them and to the whole expert committee.

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CHAPTER 4

Dynamic loads in new engineering codes for railway bridges in Europe and Spain

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ABSTRACT: Bridges and viaducts for high speed trains are subject to demanding dynamic loads. In addition to the classical effect of the moving (single) load, much larger and potentially dangerous effects arise from dynamic resonance, for speeds above 200 km/h. The classical methods for evaluation of dynamic impact factors, reflected in the codes of practise existing until recently, do not cover this possibility of resonance. The design of such structures requires dynamic calculations which are the object of this paper. We discuss briefly available methods for dynamic analysis, as well as the new codes IAPF [9] and Eurocode 1 for actions on bridges [10]. One of the key aspects which is desirable for the new lines is their qualification for interoperability, so that all possible present and future European high speed trains may use them [19]. The proposal for this is covered in [10, 9] through the new High Speed Load Model HSLM, whose background is discussed here. Finally, some results obtained by our group are presented for high speed traffic loads on bridges. These studies focus on the evaluation of the bridge-vehicle interaction in bridges and a discussion and proposal for dynamic uplift dynamic effects. These topics originate from issues in the application of the new regulations for high speed lines, and are oriented toward being of practical use to designers of railway bridges.

1 INTRODUCTION

Currently one of the main efforts in civil engineering in Spain is dedicated to the new high speed railway lines. These will provide an efficient transport links between cities at a nationwide level and with other European nations.

This important engineering activity highlights one of the main structural issues associated specifically to the design of bridges and structures in railway lines: the dynamic effects due to moving loads from train traffic. This has been considered since the early stages of railways as one of the main design requirements for the structures. The basic dynamic response for a moving load on a simply supported beam is known from classical solutions (among others) by Timoshenko [1]. Further studies have been developed among others by Fryba [2] and Alarcón [3, 4].

The classical design codes existing up to now [5, 6, 7] for design of railway bridges consider the dynamic response through an *impact factor*, which represents the increase in the dynamic response with respect to the static one for a *single moving load*.

However, high speed railway lines pose dynamic problems of higher order, due to the possibility of resonance from high speed traffic. This appears at speeds above 200 km/h, considering the typical distances between axles in railway coaches and the main eigenfrequencies of bridges. Resonance occurs when the excitation frequency coincides with that of the fundamental vibration mode of the bridge. This may be quantified through the so called *wavelength* of excitation,

$$\lambda = \frac{v}{f_0}, \tag{1}$$

where f_0 is the first natural frequency of deck vibration and v the train speed. Resonance occurs when the characteristic length D_k of separation between axles coincides with a multiple of the said wavelength:

$$\lambda = \frac{D_k}{i}, i = 1, 2k \Rightarrow \text{resonance.} \tag{2}$$

Within Europe a joint effort for research and study of dynamic effects in high speed lines has been carried out within the European Railway Research institute (ERRI) by subcommittee D214 [13]. These and other findings have been included in the recently drafted engineering codes [8], [10] and [9].

A further aspect which must also be considered is the convenience that the railway lines not be restricted to their use by a limited family of trains. On the contrary, they should allow the transit of all possible high speed trains, enabling *interoperability* of the infrastructure [19]. This issue is not only essential from a social and economical point of view, but has also other implications, following the new European directive to separate the business of the infrastructure with that of the transport operators. Only with adequate interoperability conditions can this be achieved.

Hence in principle all European high speed train types should be considered for the design and the corresponding dynamic analyses. The current high speed train types described in [9] and [10] vary widely as to distance between axles, coach lengths, etc. They may be classed into three categories: articulated trains (one bogie between coaches), conventional trains (two bogies per coach), and regular trains (one axle between coaches). However, this strategy of performing dynamic analyses for all train types is not only cumbersome and time consuming, but it also does not guarantee the validity of possible new trains which may appear in the future during the life of the structure. One of the most valuable results of the work of ERRI D214 committee has been the establishment of a *High Speed Load Model (HSLM)* [14]. This model comprises a family of (fictitious) articulated trains whose dynamic effect has been proved to be an envelope of all current trains as well as those foreseen within an agreed set of *interoperability criteria*. These aspects are discussed in section 3.

Structural engineers responsible for design of bridges are well used to static analysis for checking service and ultimate limit states (SLS, ULS) of structures. A discretised model of a structure, say by Finite Element Methods (FEM), will give rise to a set of algebraic equations, either linear or nonlinear. However, dynamic analysis on the same discretised model will originate a set of ordinary differential equations in time, whose interpretation may not be so intuitive for engineers. These must be solved either by direct integration of the complete model or by modal analysis of a selected subset of vibration modes. Traditional engineering practice for railway bridges requires only static analysis and use of an impact factor. However, the enhanced dynamic effects in high-speed railway bridges often require dynamic analyses. In summary, the models available in practice for consideration of dynamic effects are, in terms of increasing complexity: 1) impact factor (section 2.1); 2) Dynamic train signature (section 2.3); 3) Moving load dynamic analysis (section 2.2); and 4) Vehicle–structure interaction dynamic analysis (section 4).

The consideration of the vehicle–structure interaction models discussed in section 2.4 results in a reduction of the effects, due to the existence of mechanisms which permit energy dissipation (dampers) or systems which interchange energy between structure and vehicle (suspension springs). For non resonant situations or statically redundant bridges, the interaction effects are not usually relevant in the calculation, being sufficient in these cases to consider moving load models. However, for isostatic decks with short spans (10 m–30 m), significant resonant effects appear with high accelerations, and often these constant load models yield results above the design limits. With vehicle–structure interaction models a significant reduction of these results may be obtained (section 4).

In the first part of this paper we present a description of the basic features of the calculation methods available for dynamic analysis of railway bridges subject to traffic loads. Following a summary of the methods prescribed in the new drafts of codes IAPF and Eurocode 1 is done. Finally some research results for specific problems obtained by our group are presented.

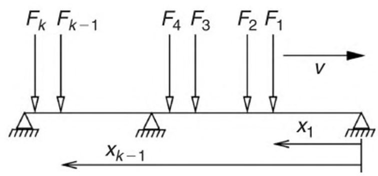

Figure 1. Model for train of moving loads.

2 DYNAMIC ANALYSIS METHODS

2.1 *Impact factor* Φ

The basic method followed up to now in the existing engineering codes for railway bridges [7, 6, 5] has been that of the impact factor, generally represented as Φ. As has been discussed previously in section 1, such coefficient represents the dynamic effect of (single) moving loads, but does not include resonant dynamic effects.

The dynamic increment for a single moving load at speed v on an ideal bridge (i.e. without track or wheel irregularities) is evaluated in [7] to be covered by the following expression:

$$\varphi' = \frac{K}{1 - K + K^4}; \quad K = \frac{\lambda}{2\mathrm{L}_\Phi}, \tag{3}$$

where L_Φ is the equivalent span of the element under study and $\lambda = v/f_0$ the wavelength of excitation. The value of the dynamic increment reaches a maximum value of $\varphi'_{\text{max}} = 1.32$, for $K = 1.76$. The final impact factor takes into account additionally the effect of irregularities through an additional term (φ''):

$$\Phi \geq \max(1 + \varphi' + \varphi'') \cdot \frac{E_{\text{sta,real}}}{E_{\text{sta,UIC71}}}, \tag{4}$$

where the factor $E_{\text{sta,real}}/E_{\text{sta,UIC71}}$ indicates that the factor will be applied not to the real trains but to the load model UIC71 [22] whose loads are an upper envelope and generally much greater than high-speed passenger trains. The impact factor so defined provides an envelope of dunamic effects:

$$\Phi \cdot E_{\text{sta,UIC71}} \geq E_{\text{dyn,real}}. \tag{5}$$

The applicability of impact factor Φ is subject to some restrictions, involving bounds for f_0 as well as a maximum train speed of 200 km/h [6]. In these cases a dynamic analysis is necessary.

2.2 *Dynamic analysis with moving loads*

The general procedure for dynamic calculation involves the time integration of the dynamic equations for the structure, under a train of moving loads (Fig. 1) representative of each axle of a given train. After discretisation of the structure, say by use of FEM, in the linear case the classical differential equations of structural dynamics are obtained,

$$\mathbf{M}\ddot{\mathbf{u}} + \mathbf{C}\dot{\mathbf{u}} + \mathbf{K}\mathbf{u} = \mathbf{f}(t), \tag{6}$$

where $\mathbf{M}$ is the mass matrix, $\mathbf{C}$ the damping matrix, $\mathbf{K}$ the stiffness matrix, $\mathbf{f}(t)$ the external load vector, and $\mathbf{u}$ the vector of (unknown) nodal displacements.

By means of the direct integration of the model, the complete system (6) of N degrees of freedom would be solved for each time step; the equations are generally coupled, and therefore must be solved simultaneously. This procedure is also valid when nonlinear effects are to be included in the response; in this case the elastic internal forces and viscous damping forces from the previous expression should be replaced by a general term (nonlinear) of the type $\mathbf{f}^{\text{int}}(\mathbf{u}, \dot{\mathbf{u}}, \dots)$.

Alternatively, a reduction of degrees of freedom may be performed through modal analysis. Modal reduction reduces substantially the number of equations to integrate, and may be performed through an approximate numerical procedure to obtain the eigenmodes of vibration. This capability is provided by most commercial finite element programs. Alternatively the modal reduction may be achieved through an analytical (closed form) calculation, for certain cases of simple structures. It is generally accepted that modes with frequencies higher than 30 Hz will provide negligible effects and are not included in the analysis.

In general it is far more efficient to integrate the reduced modal equations. The first step is to obtain the eigenmode shapes and associated eigenfrequencies. For the trivial example of a simply supported bridge, the eigenmodes may be expressed analytically [12] as $\phi_i(x) = \sin(i\pi x/l)$, with associated eigenfrequencies $\omega_i = (i\pi)^2\sqrt{EI/(\overline{m}l^4)}$. For this simple case, it is generally sufficient to consider a single vibration mode; this way the problem is reduced to a dynamic equation with one degree of freedom. However, for an accurate evaluation of section resultants or reactions a larger number of modes may be necessary, as discussed in [17]. For more complex structures it is also necessary to consider more vibration modes.

Once the vibration modes are known, the basic response of each mode to a single moving load F or to a complete load train $F_k, k = 1 \dots n_{ax}$ (Fig. 1) may be evaluated. This may be assembled as the superposition of the responses for each individual load F_k in the following manner:

$$M_i\ddot{y}_i + 2\zeta_i\omega_i M_i\dot{y}_i + \omega_i^2 M_i y_i = \sum_{k=1}^{n_{\text{ax}}} F_k\langle\phi_i(vt - d_k)\rangle. \tag{7}$$

In the above equation, $\phi_i(x)$, M_i and ω_i are respectively the modal shape, modal mass and eigenfrequency for eigenmode i; y_i is the modal amplitude, ζ_i the damping ratio, and $\langle\phi(\bullet)\rangle$ represents a bracket notation with the following meaning:

$$\langle\phi(x)\rangle = \begin{cases} \phi(x) & \text{if } 0 < x < l \\ 0 & \text{otherwise} \end{cases} \tag{8}$$

Except for particular cases of simple structures the above equations (6), (7) must be solved numerically by finite element programs. These provide an efficient method for calculation in arbitrary structures. Adequate procedures for preprocessing (definition of load histories for all individual axles) and postprocessing are necessary for their practical use in engineering design work [18].

It is important to consider also ELS dynamic limits [20, 21] (maximum acceleration, rotations and deflections, etc.), which are often the most critical design issues in practice. Accelerations must be independently obtained in the dynamic analysis. In the example shown in Figure 5 both maximum displacements and accelerations are obtained independently and checked against their nominal (UIC71) or limit values respectively.

2.3 *Simplified models based on dynamic train signature*

An alternative which may be useful in some cases are the so-called *dynamic train signature* models. These develop the response as a combination of harmonic series, and establish an upper bound of this sum, avoiding a direct dynamic analysis by time integration. In counterpart their application is limited to *simply supported bridges*, which can be represented dynamically by means of a single harmonic vibration mode. They have been developed and used for a number of years within SNCF, and their basic description may be found in [13].

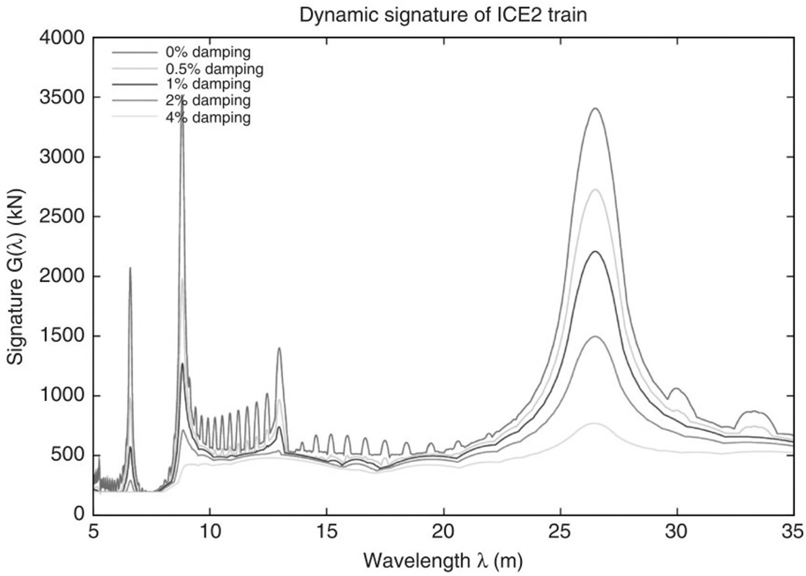

Figure 2. Dynamic signature of ICE2 train for different damping ratios.

The *dynamic signature* of a train may be interpreted as a function which characterises its dynamic effect on a given railway bridge. The models of this type proposed are:

- DER, based en the Decomposition of the Resonance Excitation;
- LIR, Simplified method based on the Residual Influence Line;
- IDP, A slightly modified version of LIR method with improved accuracy [11].

All these methods furnish an analytical evaluation of an upper bound for the dynamic response of a given bridge, as a product of three terms: a constant term, a *dynamic influence line* of the bridge, and a *dynamic signature* of the train. Let us take as an example the LIR method for evaluating the maximum acceleration. This procedure is based on the analysis of the residual free vibrations after each individual single load crosses a simply supported bridge. The acceleration Γ at the centre of the span is given by:

$$\Gamma = \frac{1}{M} \cdot A(K) \cdot G(\lambda), \tag{9}$$

where

$$A(K) = \frac{K}{1-K^2}\sqrt{e^{-2\varsigma\frac{\pi}{K}} + 1 + 2\cos\left(\frac{\pi}{K}\right)e^{-2\varsigma\frac{\pi}{K}}}, \tag{10}$$

$$G(\lambda) = \max_{i=1}^{N}\sqrt{\left[\sum\nolimits_1^i F_i \cos(2\pi\delta_i)e^{-2\pi\varsigma\delta_i}\right]^2 + \left[\sum\nolimits_1^i F_i \sin(2\pi\delta_i)e^{-2\pi\varsigma\delta_i}\right]^2}, \tag{11}$$

being M the mass of the bridge, F_i the N axle loads, $\delta_i = (x_i - x_1)/\lambda$ the normalized distance of each axle to the head of the train and ζ the damping ratio.

The term $G(\lambda)$ defined in equation (11) is the *dynamic signature* referred to above. It depends only on the distribution of the train axles and the damping rates. Each train has its own dynamic signature, which is independent of the mechanical characteristics of the bridges. As an example, Figure 2 represents the dynamic signature of train ICE2, for different values of damping.

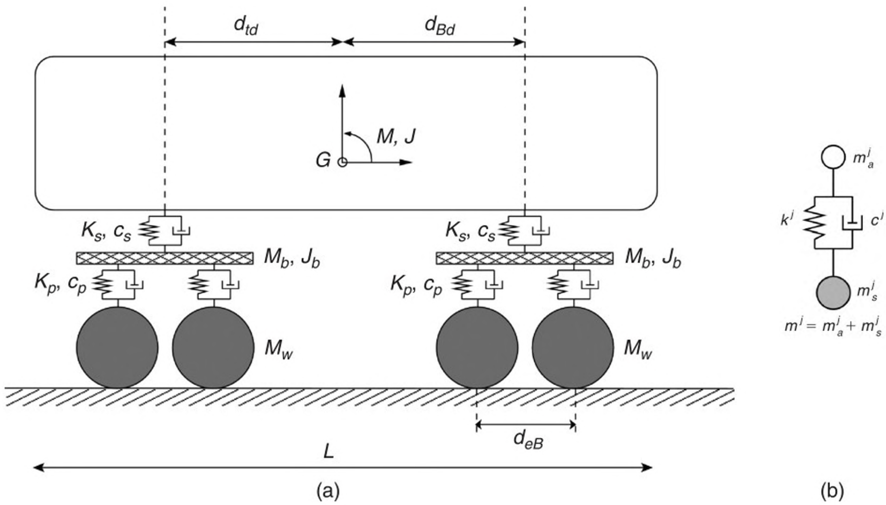

Figure 3. a) Complete vehicle-structure interaction model; b) simplified interaction model.

The term $A(K)$ defines a function of K (itself dependent on speed v), called the bridge *dynamic influence line*. It depends solely on the span L, the first natural frequency (f_0) and damping (ζ). It does not depend on the characteristics of the train. Separating this contribution from the bridge and the dynamic signature of the train $G(\lambda)$, it is possible to determine easily the critical parameters of span and wavelength for which the dynamic response of the bridge is maximum.

As has been said before, these dynamic signature methods have been developed in principle for simply supported, isostatic bridges. However, some studies have been carried out which in some cases extend their applicability to certain classes of redundant structures. For instance, Liberatore [15] has developed dynamic signature methods to establish the modal aggressivity of continuous decks with 2 spans.

2.4 *Dynamic analysis with vehicle-structure interaction*

The consideration of the vertical motion of the vehicles with respect to the bridge deck allows for a more realistic representation of the dynamic overall behaviour. In these models the train is no longer represented by moving loads of fixed value, but rather by point masses, bodies and springs which represent wheels, bogies and coaches. In some cases these models may have a non negligible influence on the dynamic response of the bridge.

A general model for a conventional coach on two bogies is shown in Figure 3a, including the stiffness and damping (K_p,c_p) of the primary suspension of each axle, the secondary suspension of bogies (K_s,c_s), the unsprung mass of wheels (M_w), the bogies (M_b,J_b), and the vehicle body (M,J). Similar models may be constructed for articulated or regular trains.

The level of detail in the above model is not always necessary. In this work we employ a simplified model in which for each axle only the primary suspension, equivalent unsprung and sprung masses are considered (Fig. 3b). For a train of k axles, there will be k such elements (Fig. 4). In this model each axis is independent from the rest, thus neglecting the coupling provided by the bogies and vehicle box. Further details are described in [11].

The model thus obtained considers a degree of freedom for each mode of the structure and an extra one for each interaction element. The equation for each mode ($i = 1, \ldots n$) is

$$M_i\ddot{q}_i + C_i\dot{q}_i + K_iq_i = \sum_{j=1}^{k}(m^jg + m_a^j\ddot{y}^j)\langle\phi_i(d_{\mathrm{rel}}^j)\rangle \tag{12}$$

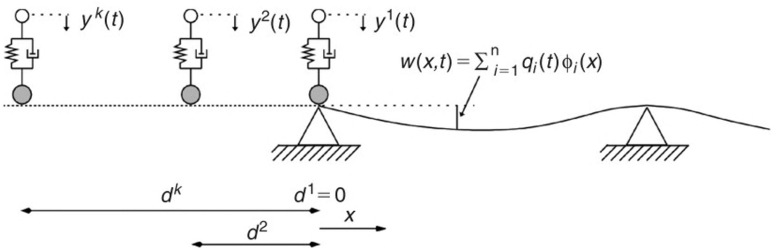

Figure 4. Representation of axle loads, according to the vehicle-structure interaction simplified model: definition of variables.

And the equation for each interaction element ($j = 1, \ldots, k$) is:

$$m_a^j \ddot{y}^j + k^j \left[y^j - \sum_{i=1}^{n} q_i \langle \phi_i(d^j \mathrm{rel}) \rangle \right]$$
$$+ c^j \left[\dot{y}^j - \sum_{i=1}^{n} \dot{q}_i \langle \phi_i(d^j_{\mathrm{rel}}) \rangle - \sum_{i=1}^{n} q_i v \langle \phi_i'(d^j_{\mathrm{rel}}) \rangle \right] = 0 \tag{13}$$

In the above equations the notation $\langle \phi(\bullet) \rangle$ defined previously (8) has been employed. Additionally, $d^j_{rel} = vt - d^j$ represents the relative position on the bridge for each element j. Finally, these equations may be solved in time by standard numerical integration schemes in structural dynamics, such as trapezoidal or HHT rules.

An application of dynamic calculations using moving loads and simplified interaction models is shown in Figure 5. A considerable reduction of vibration is obtained in short span bridges under resonance by using interaction models. This may be explained considering that part of the energy from the vibration is be transmitted from the bridge to the vehicles. However, only a modest reduction is obtained for non-resonant speeds. Furthermore, in longer spans or in continuous deck bridges the advantage gained by employing interaction models will generally be very small. This is exemplified in Figure 9, showing results of sweeps of dynamic calculations for three bridges of different spans. As a consequence it is not generally considered necessary to perform dynamic analysis with interaction for design purposes.

3 HIGH SPEED REAL TRAINS AND INTEROPERABILITY

The new European high-speed lines should ensure interoperability for rolling stock , that is all high speed trains from other European nations should also be able to circulate among these lines [19]. From the point of view of structural requirements on bridges the static strength is assured by the static load model UIC71. The dynamic performance must be assured by a set of dynamic analyses that covers all possible present (and future) trains.

European high speed trains may be classified into three different types (Fig. 6):

1. *Articulated trains*: each two coaches share one bogie between them. This type includes THALYS, AVE and EUROSTAR.
2. *Conventional trains*: each coach has two bogies. This includes ICE2, ETR-Y, VIRGIN.
3. *Regular trains*: coaches are supported not on bogies but on single axles in the junction between each two of them. This is the case of TALGO.

To ensure dynamic performance not only for the above trains but also for their possible variations and future developments through dynamic analysis ("brute force method") would be extremely costly as well as of doubtful efficiency. Small variations in the configuration of a given train may

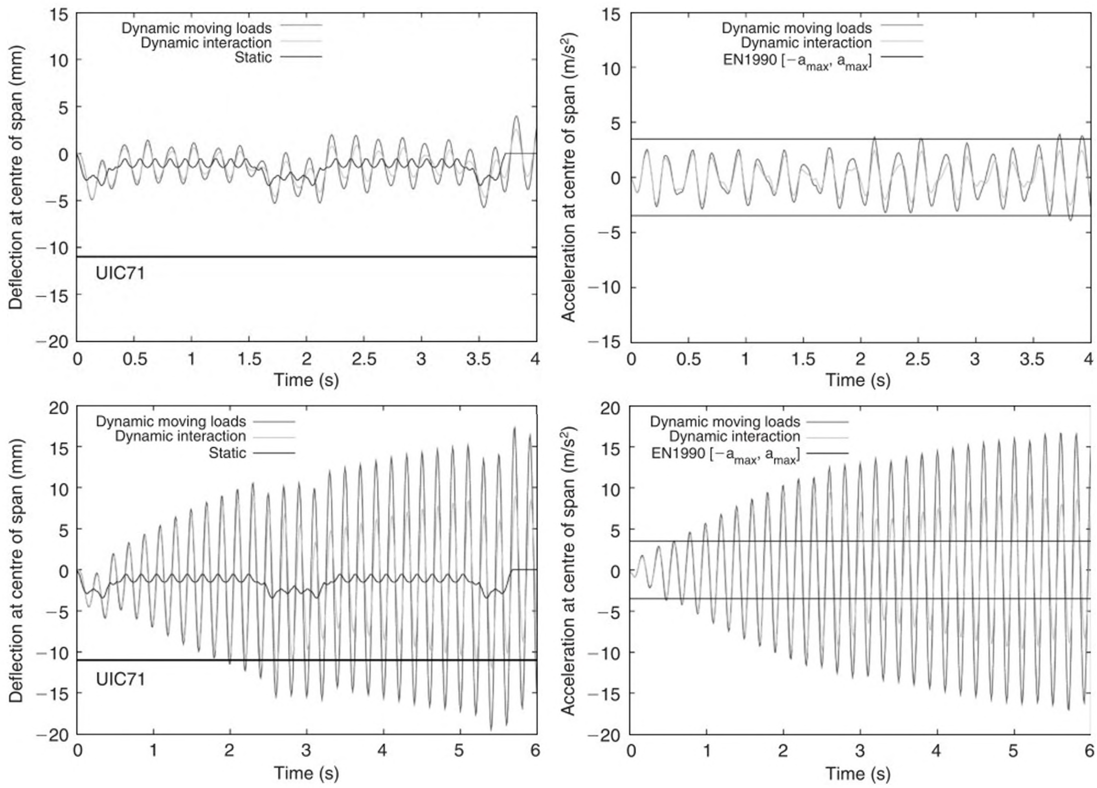

Figure 5. Calculations for simply supported bridge from ERRI D214 (2002) ($L = 15$ m, $f_0 = 5$ Hz, $\rho = 15000$ kg/m, $\delta_{\mathrm{UIC71}} = 11$ mm), with TALGO AV2 train, for non-resonant (360 km/h, top) and resonant (236.5 km/h, bottom) speeds, considering dynamic analysis with moving loads and with train-bridge interaction. Note that the response at the higher speed (360 km/h) is considerably smaller than for the critical speed of 236.5 km/h. The graphs at left show displacements, comparing with the quasi-static response of the real train and the UIC71 model, and those at right accelerations, compared with the limit of 3.5 m/s^2 [EN1990 (2004)].

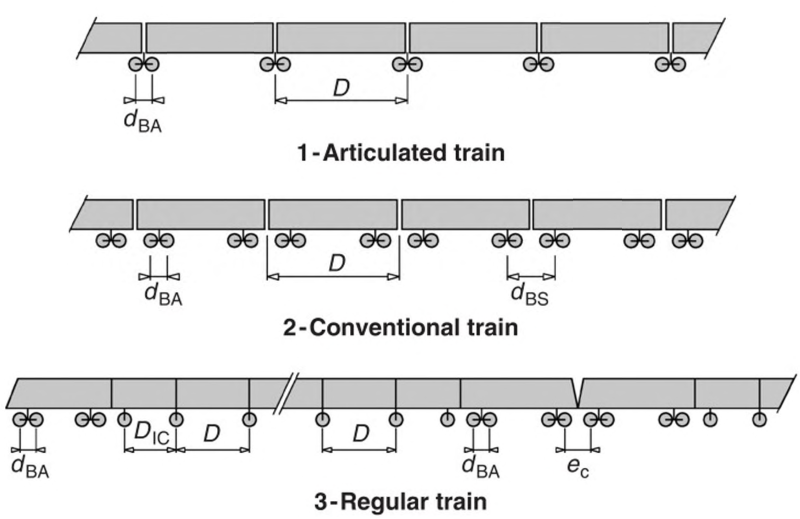

Figure 6. Different types of high-speed trains, according to Eurocode EN1991-2 [10].

influence significantly the resonant peaks, making it extremely difficult to assure the fulfillment of the dynamic performance interoperability conditions.

The concept of train signature (section 2.2) is very useful for the purpose of obtaining a dynamic envelope. Figure 7 shows the dynamic signature (common for LIR and DER methods) obtained

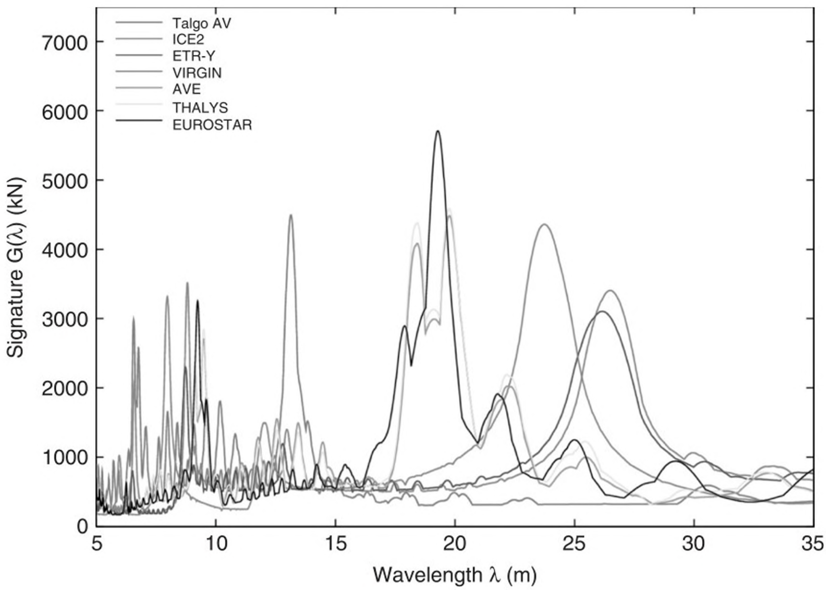

Figure 7. Dynamic signatures (zero damping) for real European high-speed trains.

Table 1. Characteristics of HSLM-A universal trains. ([14], [10], [9], [19]).

	HSLM-A
Type	Articulated
Total length	≈400 m
Coach length D	18 m–27 m
Axle point load	170 kN–210 kN
Bogie axle spacing d	2.0 m–3.5 m
Head and tail locomotives	Yes

for the most common current European high-speed trains. An envelope of these signatures may be easily obtained.

The task of developing a High Speed Load Model (HSLM) which would ensure interoperability conditions was performed by ERRI D214.2 [14], which first drafted envelopes of DER signatures for all current high-speed trains and their possible variations. Following, a family of fictitious articulated trains (Universal trains) was devised ensuring that their signature envelope effectively covered the signatures of all real trains. Table 1 summarises the characteristics of HSLM-A family of universal trains. A further family HSLM-B must be used for bridges with span $L < 7$ m.

Dynamic analysis with HSLM model hence requires the analysis for the 10 universal dynamic trains at all possible speeds, up to 1.2× the maximum permitted speed. In Figure 8 the envelope of real HS strains together with the envelope of HSLM-A trains are drawn, showing the fact that HSLM-A are indeed an envelope for dynamic effects of real trains.

Further to the above envelopes, a procedure is also defined in [14] and [10] for simple bridges which allows determining the (most aggressive) critical universal train and associated speed. In such cases the dynamic analysis is simplified.

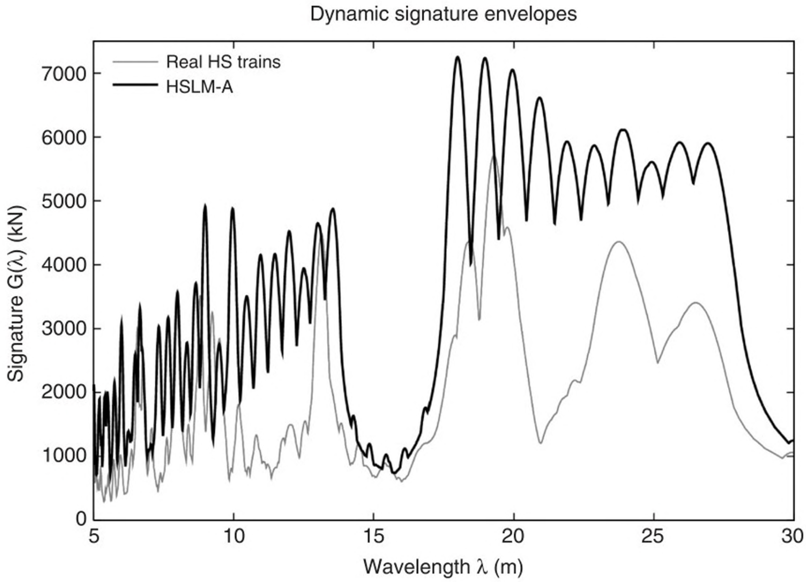

Figure 8. Envelopes of dynamic signatures (zero damping) for real European high-speed trains and HSLM-A load model trains.

4 APPLICATION EXAMPLE: INFLUENCE OF BRIDGE SPAN

Generally resonance may be much larger for short span bridges. As a representative example, Figure 9 shows the normalised displacement response envelopes obtained for ICE2 train in a velocity sweep between 120 and 420 km/h at intervals of 5 km/h. Calculations are performed for three different bridges, from short to moderate lengths (20 m, 30 m and 40 m). The maximum response obtained for the short length bridge is many times larger that the others. The physical reason is that for bridges longer than the vehicle length at any given time several axles or bogies will be on the bridge with different phases, thus cancelling effects and impeding a clear resonance. We also remark that for lower speeds in all three cases the response is approximately 2.5 times lower than that of the much heavier nominal train UIC71. Resonance increases this response by a factor of 5, thus surpassing by a factor of 2 the UIC71 response.

Another well-known effect which will not be followed here due to lack of space is the fact that dynamic effects in indeterminate structures, especially continuous deck beams, are generally much lower than isostatic structures [11]. The main reason for this is that for simply supported bridges only one fundamental mode dominates the response; whereas in continuous beams several modes have an effective participation, which cancel each other partially under the moving loads.

5 EVALUATION OF THE DYNAMIC VEHICLE-STRUCTURE INTERACTION

The object of this application is to evaluate the effective reduction which is obtained, with respect to the dynamic analysis made without considering the vehicle-structure interaction, such as the models based on series of harmonics or models of moving loads, more common in engineering practice.

The calculations are based on a modal analysis considering only first mode of vibration, without shear deformation. A model of moving loads is compared to the model with interaction proposed in section 2.4. Time integration has been carried out using the trapezoidal rule. A family of simply

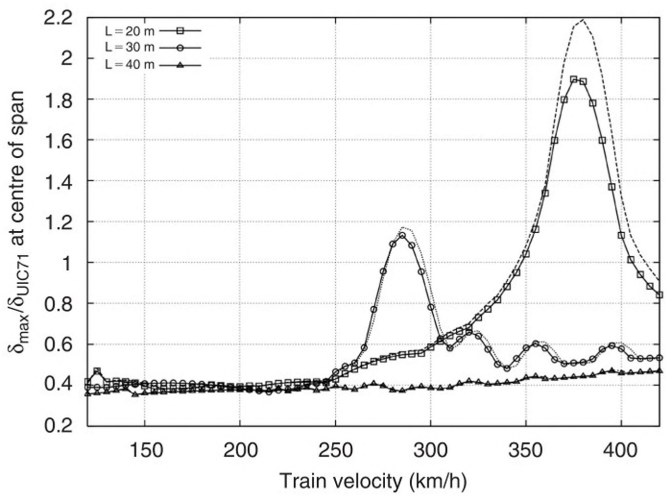

Figure 9. Normalised envelope of dynamic effects (displacement) for ICE2 high-speed train between 120 and 420 km/h on simply supported bridges of different spans ($L = 20$ m, $f_0 = 4$ Hz, $\rho = 20000$ kg/m, $\delta_{UIC71} = 11.79$ mm, $L = 30$ m, $f_0 = 3$ Hz, $\rho = 25000$ kg/m, $\delta_{UIC71} = 15.07$ mm and $L = 40$ m, $f_0 = 3$ Hz, $\rho = 30000$ kg/m, $\delta_{UIC71} = 11.81$ mm). Dashed lines represent analysis with moving loads, solid lines with symbols models with interaction. Damping is $\zeta = 2\%$ in all cases.

supported bridges of spans (L) between 10 and 40 m has been considered, with characteristics following the catalogue of isostatic bridges from [13]. The speed sweep is (120–420) km/h, with $\Delta v = 2.5$ km/h. The trains used are the ICE2, EUROSTAR and TALGO AV, defined in [9], with damping rates $\zeta = 0.5\%$, 1%, 1.5%, 2.0%. Further technical details of the numerical model are contained in [11].

The analysis results, as was predictable, show a significant reduction of the maximum displacements and accelerations for models with interaction. Some of the results obtained are included in Table 2.

In view of the results shown, one may conclude in first place that the moving load models generally overestimate slightly the resonant response in accelerations and displacements of an isostatic structure; in extreme cases, the interaction models can reduce the maximum acceleration values in isostatic bridges up to 45% respect to acceleration obtained with moving load models. However, as was shown in Figure 9, in practical cases the reduction is generally more modest. Furthermore, in non-resonant situations the reduction is virtually negligible. It is also observed that the reduction of the response decreases when damping ratio or the bridge span increases.

Additionally, the dynamic response reduction, for the same hypothesis of span and damping, is greater for accelerations than for displacements, and the reduction increases as the line design speed is increased.

6 DYNAMIC UPLIFT

We discuss here some results for evaluating dynamic uplift effects. Some of these have been considered for drafting the code [9]. Under some circumstances these may be relevant from a structural point of view. A typical example is the verification of bridge piers in a continuous deck bridge, for which the limiting case is the minimum vertical loads simultaneously with maximum horizontal loads (centrifugal and wind mainly). This aspect is not addressed directly in [10], where an *unloaded train* is proposed for these scenarios. The results shown here summarize those described with greater detail in [17].

Table 2. Percent reduction for maximum acceleration and displacement for vehicle-structure interaction model as compared to moving load model. Max $V^{\text{line}} = 220$ km/h and 375 km/h.

220 km/h	$\zeta = 0.5\%$		$\zeta = 1.0\%$		$\zeta = 2.0\%$	
L (m)	disp.	accel.	disp.	accel.	disp.	accel.
5	−25%	−35%	−15%	−25%	−10%	−20%
10	−30%	−35%	−20%	−25%	−10%	−15%
15	−25%	−45%	−15%	−35%	−5%	−20%
20	−10%	−20%	−5%	−15%	0%	−10%
25	−10%	−35%	−5%	−25%	0%	−10%
30	0%	−15%	0%	−5%	0%	−0%
40	0%	−10%	0%	−5%	0%	−5%

375 km/h	$\zeta = 0.5\%$		$\zeta = 1.0\%$		$\zeta = 2.0\%$	
L (m)	disp.	accel.	disp.	accel.	disp.	accel.
5	−25%	−35%	−15%	−25%	−10%	−20%
10	−30%	−35%	−25%	−25%	−15%	−15%
15	−30%	−45%	−20%	−35%	−10%	−20%
20	−20%	−20%	−15%	−20%	−10%	−15%
25	−20%	−35%	−15%	−25%	−5%	−15%
30	−10%	−15%	−5%	−15%	−5%	−10%
40	−5%	−10%	0%	−10%	0%	−5%

As a result of interpreting the dynamic response as oscillations around a quasi-static state it is possible to obtain bounds for maxima and minima, computed from the said static response and the amplitude of oscillation. Figure 10 shows the vertical reaction in a pier between two (simply supported) spans in a real bridge (Tajo river), computed for three different cases with the Eurostar train. Details of the structure and of analysis model may be found in [16]. Two of these cases are dynamic results for a speed of 225 km/h which was shown to produce resonance, with a moving load model and with a vehicle-structure interaction model. Additionally, the quasi-static low-speed (20 km/h) results are superposed on to the previous cases (these are previously scaled in pseudo-time in order to correspond with the dynamic cases).

The above results show that the dynamic vibration may be interpreted as a dynamic effect $\pm\Delta E_{din}$ which is superposed on the quasi-static one, E_{sta}. The maximum dynamic effects obtained would be $E_{\max} = E_{sta} + \Delta E_{dyn}$, whereas the minima would result $E_{\max} = E_{sta} - \Delta E_{dyn}$. The time instant in the figure for which the level $E_{\min}$ shown ceases to be a lower bound corresponds to a moment at which the train has already exited the first span, which then remains in free vibration. The minimum dynamic effects correspond to unloading, that is upward reactions. Although these are significant, they would not effectively produce a lifting of the deck from the pier which would prescribe an anchorage, due to the permanent self-weight loads. However, their consideration may be necessary for some design features such as those governed by horizontal loads.

A further feature which may be observed in Figure 10 is that the model with interaction predicts results which are slightly below those of the moving load model. This was expected in a resonant scenario.

A complete set of analyses of this type has been carried out for a set of simply supported and hyper static (continuous deck) bridges, reported in [16]. In Figure 11 a representative result is shown for a continuous deck viaduct with 17 spans over river Cabra. The case shown here is for the bending moment at the centre of the first span, produced by Eurostar train at 420 km/h. This result differs in several important aspects from the previous one. Firstly, although the speed selected is that for which maximum dynamic response was obtained, it does not correspond to a resonant

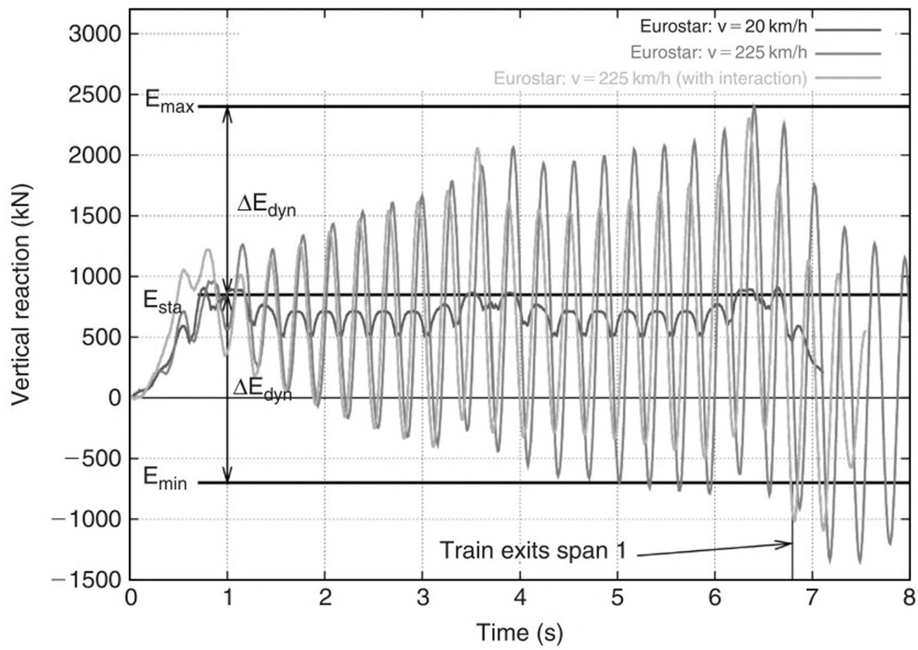

Figure 10. Time history of vertical reactions at a pier of Tajo river viaduct (simply supported spans), for Eurostar train at a speed of $v = 225$ km/h (resonant speed).

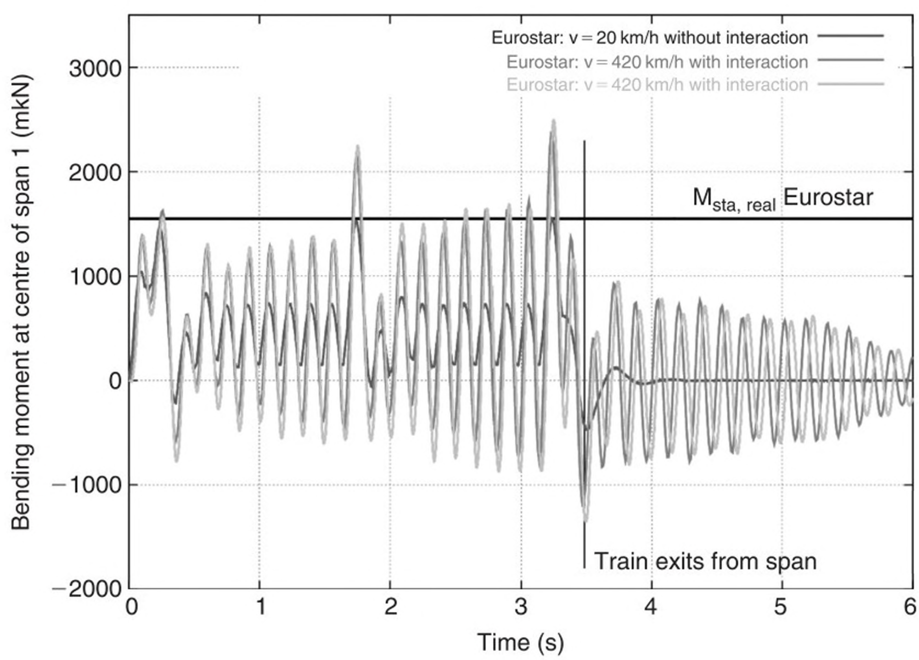

Figure 11. Time history of bending moment at the centre of the first span of the continuous deck viaduct over Cabra river, for Eurostar train at a speed of $v = 420$ km/h (speed for maximum dynamic effects).

scenario. (This is common in hyper static bridges for which resonant peaks are not so pronounced or may not be significant, as numerous competing vibration modes take part in the response at a given point.) As a consequence, the dynamic response has a lower relative importance relative to the quasi-static response. In other words, the quasi-static part of the response represents a greater fraction of the total maximum or minimum dynamic response. Secondly, the result for the model with interaction is also shown here. In this case the result predicted by the model with interaction turns out to be greater than the one with moving loads. This is also due to the fact mentioned above that the situation is not dominated by resonance, contrary to the results shown in section 5.

From the above results and the consideration of the complete set of results in a set of representative cases [16], [17], a proposal was drafted for a design envelope of uplift effects:

$$\Phi_{\min} = 2f_{e-}\Phi_r; \quad \Phi_{\min} \geq 0, \tag{14}$$

where $f_e = E_{sta,real}/E_{sta,LMd}$ is the ratio between the static response for real trains and that of the design static load model (UIC71 $\times\ \alpha$), and Φ_r is the real impact coefficient, defined by $E_{\max} = \Phi_r E_{sta,LMd}$. The loads for the design static load model are considerably larger than those of the much lighter passenger high speed trains, and as a result f_e normally lies between 0.25 and 0.35. Consequently, the coefficient $\Phi_{\min}$ may end up having negative values, which would represent a net dynamic uplift due to the traffic induced structural vibration. We must take into account that this net uplift must be superposed to the generally greater effects of the permanent self-weight loads, hence the deck would not really lift up from the piers.

7 CONCLUDING REMARKS

As a consequence of the work described above we summarise the following remarks:

- Dynamic effects in general and the possibility of resonance in particular require a dynamic analysis for the design of high speed railway bridges.
- The consideration of dynamic vehicle-structure interaction leads to more realistic predictions, in the case where adequate data from the trains are available to build such models. The structural response predicted is somewhat lower to that of moving load models for resonant scenarios. However, they are generally not warranted for practical design purposes.
- Simplified models based on the dynamic train signature which provide upper bounds for dynamic effects, although of limited applicability, may provide an interesting insight into the dynamic response of bridges. Moving load finite element models or even vehicle-structure interaction models for more special cases provide a general methodology.
- Hyper static continuous deck bridges lead generally to a less marked resonance, although a dynamic analysis is still necessary for them. In practice, HSLM models for interoperability of railway lines are adequate bounds of the dynamic effects in the cases studied.
- It may be necessary to consider both signs of dynamic effects, including also the dynamic uplift which may be significant in some design scenarios. This may be done through specific design provisions or through a special unloaded train.

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CHAPTER 5

The Italian high speed network: Design and construction of the reinforced concrete bridges

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ABSTRACT: This paper deals with the main aspects involved in the design and construction of the reinforced concrete bridges of the new Italian high speed Network. At the beginning, the authors present the main features of the new high speed/capacity railway network and the strategic choices from the infrastructural and structural points of view. The technical national code [1] for the design of railways bridges and the direct consequences due to the strategic choices are also illustrated. The state of art in Italy with regard to prestressed concrete bridges and viaducts for high-speed railways is then analysed. More than 90% of bridges and viaducts of the new high-speed lines are realised with simply supported spans of prestressed concrete decks, so special attention will be paid on their typical structural solutions. Main concepts of durability of structures involved in design of railway bridges are focused. The authors present the activities of modelling and testing on prototypes, usually preliminary to construction, and load testings. Finally, construction processes are described, taking examples from the latest viaducts with prestressed concrete decks currently under construction. At the end, future trends are presented on the basis of the critical evaluation of the present structural solutions.

1 INTRODUCTION

This paper deals with an analysis of the Italian high speed network: attention will be paid in our analysis only to design and construction of the reinforced concrete bridges with a presentation of the state of art with regard to prestressed concrete deck for simply supported spans.

This overview is possible due to the role played by ITALFERR S.p.A., the engineering company of the Italian state railways, that is charged with technical and site control on the design and building of the new high speed lines.

Since 1994 new high speed network is under construction in Italy, in order to increase the capacity of the actual railway operating lines, nearly doubling the actual number of trains running daily, and in order to decrease travel time. This will be achieved with new high capacity/speed railways, connecting through two main axis, North to South and East to West, all main cities of our Peninsula.

To give a couple of examples, it will be possible to travel from Milano to Roma in 3 hours (now 4 h 30′) and from Torino to Napoli in 5 hours (now about 9 hours). About 650 km of these new lines are under construction and partially already built (see blue lines in Fig. 1) and works are expected to be completed in 2009 [2]. These lines will then connect, with the operating line Roma – Firenze, Torino to Napoli.

The new lines are designed according to the national code for the design and construction of railways bridges [1], and to the most advanced technological standards in order to achieve the safest conditions of service, speed and interoperability with operating railways and with the European high-speed lines of railway transportation of passengers and freight.

Figure 1. Italian New High Speed Network: lines under design, construction and the operating line Roma-Firenze, built in the 70's–80's for a design speed of 250 km/h [2].

A preliminary analysis of the results of the periodical inspection of old railway lines and the Roma-Firenze railway line, built in the 70s–80s, led in the 90s to new concepts of design devoted to care for durability of structures as bridges and viaducts. These results together with a study of International Standards for high speed lines and deeply investigated new technologies of the last ten years have lead to the design of the Italian new high-speed lines.

Bridges and viaducts compose almost 18% of the new Italian high-speed lines and about 90% of these are made of simply supported prestressed concrete decks. Simply supported composite steel and concrete spans, few continuous bridges and some special structures (arch bridges and the cable-stayed bridge over Po River [3]) compose the remaining part.

2 STRATEGIC CHOICES

Traffic analysis carried out on existing Italian railway network at the end of the 1980 remarked the following needs: quadrupling the main passenger transport routes, upgrading and increasing freight transport, reducing time of travel for passengers trains, integrating Italian network with European network.

So, the first infrastructural choice was the realization of a new mixed passenger/freight high-speed network with a close integration with existing lines and with interchange centres (interports, ports, airports).

The close integration with the existing conventional network will produce an increase of freight transport capacity on the "historical" lines, clearing the existing network from the long distance passengers traffic, and an increase of freight traffic using new lines during specific time bands (usually at night).

The choice of a mixed traffic meant low ruling gradients (less than 12‰) and heavy design loads (SW0/SW2) adopted in the new Italian standard for railway bridges [1]. For this reason, the standard was rewritten in 1995, then revised in 1997. The old standards had been written 50 years before and were related only to conventional lines. The Italian standard for railway bridges [1] has introduced LM71 (passengers traffic as showed in leaflets UIC 702 and 776-1), SW/0 and SW/2 (heavy traffic) models of loads according to ENV 1991-3: Actions on structures, Part 3: Traffic loads on bridges (Ed. 1996).

These new standards are in perfect agreement with the European Technical Standards for Interoperability of the trains in the European High-speed Network.

One of the main prescriptions asks for a structural design respectful to all prescriptions for seismic areas (at least III category – the minimum considered in the 1996 Italian seismic code), even in no-seismic areas, apart from Sardinia. A proper standard was written and recently revised for the design of railway bridges to be built in seismic area [4]. It deserves to be pointed out that many no-seismic areas became recently seismic, in the latest proposals of codes, giving interesting confirmation to the conservative railways code approach.

According to general seismic design principles, the adoption of special rules and technical details is requested to guarantee a minimum ductility of the structure, and it has direct consequences on the care for the details of reinforcement design of piers and foundations.

Two main characteristics of the applied national code [1] are the concepts of train-track-structure dynamic interaction and train-rail-structure static interaction.

The dynamic interaction analysis is evaluated to check the safety of the train and the comfort of the passengers, with an analysis of the following parameters: all decks for high-speed railway must respect the limit value of 2.5 as maximum dynamic amplification of static deflection ("impact factor" $\varphi_{real} = \delta_{dyn}/\delta_{stat}$) and the value of the vertical acceleration at deck midspan, induced by real trains running at different speed (from 10 km/h up to 1.2 maximum speed of the line), must be lower than 3.5 m/s^2.

For standard simply supported spans or continuous bridges with total length shorter than 130 m, a simplified analysis can be adopted according to Annex A of [1], with a preliminary check of flexural – frequencies of vibration modes: the simply supported prestressed concrete decks always respect the limit values specified in [1], being first flexural mode frequency between 4 and 8 Hz.

For non-conventional structures, as arch bridges, cable-stayed bridges, etc., a "Runnability" analysis is required. The analysis has to consider all dynamic characteristics of the system: railway structures, suspension system of the vehicles, rail fasten system etc., track and wheel irregularities.

The static interaction analysis studies the effects on rail and bridge structure of the system of actions due to variation of thermal conditions in the structures, to the longitudinal forces associated to braking and traction, and to the longitudinal displacements due to vertical loads. For simply supported prestressed concrete decks, which respect a maximum length of 65 m and small variations of the longitudinal stiffness of piers and foundations, a simplified method can be adopted as indicated in Annex B of [1].

In any other case, it is necessary to analyse advanced Finite Element models to evaluate these effects. The analysis has to check rail stress limits with maximum value of compressive stress 60 MPa and maximum value of tensile stress 70 MPa, the relative displacement between deck bridge and the rail, and the forces acting on the bearings.

Also reliability under service conditions is required: comfort limit state has to be verified for a maximum midspan deflection with the load of one LM71 load model, increased with dynamic factor. This deflection must not exceed $l/2400$ for design length $l < 30$ m, $l/2800$ for $30\text{ m} < l < 60$ m and $l/3000$ for $l > 60$ m. Maximum deformability of structures under train load is checked to keep the contact rail-wheel safe and stable: deck torsion, rotation at supports and horizontal deflection have to be evaluated. The limits of deformation of the structures are similar to those pointed out in the same Eurocode, and are widely respected by common simply supported spans.

Special attention has been put in the concepts of durability of structures for railway bridges, introduced in [1]. As the subject deserves wide illustration and details, a full paragraph has been devoted to the scope.

As high-speed network is designed for the use of long welded rail, the structural system for the viaducts must avoid rail expansion devices. In all high-speed network, only along the Milano-Bologna line, for the crossing over Po River, composed by two continuous bridges and the cable stayed bridge [2], two joints in the rails have been necessary to keep the expansion length within allowable limits.

Other rules and prescriptions for design and construction are taken into consideration in [1] and are illustrated in the following paragraphs. All these are finalised to have low costs of maintenance of the infrastructure, to minimise the irregularities of the track and to reach a high performance level in the field of the durability and reliability of the system.

3 PRESTRESSED CONCRETE BRIDGES

Simply supported spans of prestressed concrete deck realised more than 90% of the new lines under construction. It is undoubtedly a traditional choice of Italian Railway Company (Ferrovie dello Stato – FS) for ordinary viaducts: to better fit with long welded rail, to avoid rail expansion devices, to ease maintenance operations and minimize maintenance costs. Besides, this solution is usually preferred in those cases when bridges have to be designed in areas with compressible soils or in river channels.

Figures and statistics of this paper are based on more than 600 km long high-speed lines already built or still under construction between Torino-Milano-Napoli, presented in Figure 1. Attention will be paid to double-track decks, with a distance between tracks of 5.0 m, designed for a train-speed of 300 km/h, and for both heavy and passengers traffic load models [1], with rails on prestressed concrete sleepers on ballast. They count nearly 2300 spans of simply supported prestressed concrete bridges, and they are composed of nearly 6400 precast beams or monolithic decks.

In order to reach this frame of prestressed concrete decks, analysis will concern the structural characteristics such as use of pre-casting, tensioning systems, bearings, expansion joints, all durability issues as multi-layer protection systems, monitoring, methods of construction and costs.

3.1 *Design*

3.1.1 *Deck*

The pre-stressed concrete elements are realised with both pre-tensioning and post-tensioning systems. The post-tensioning systems are always designed with bonded internal cables, even if Italian standard for railway bridges [1], generally speaking and under severe controls, allows also external post-tensioning. According to [1], post-tensioning cables composed by bars should be preferred for viaducts along railways with electric traction of direct current, and both solutions with cables composed by strands or bars can be used in structures for railways with electric traction of alternating current as the high-speed network.

In [1], special attention for durability and limiting or avoiding cracking of concrete is introduced: undoubtedly most limiting verifications deal with limitation of maximum compressive stresses and, especially, strong limitation of tensile stresses during construction and final conditions. In particular, no longitudinal tensile stress in PC structures is admitted, with maximum design loads and both Allowable Stress or Limit States methods of verification.

Besides, cracking of concrete must be verified towards no decompression limit state for verification under track equipment, where inspection is not possible.

The experience of existing railway lines with concrete structures with possible beginning of corrosion of the reinforcement and spalling of the concrete, which leads to easier access to the pre-stressing tendons for aggressive agents, has been translated into design prescriptions. The required concrete cover to reinforcement, tendons and pre-tensioned strands has been increased, compared to Italian standard for design of structures. Minimum concrete cover thickness is required to be 3 cm for PC decks, increased to 3.5 cm under track equipment, at least one external diameter of duct in case of post-tensioning, and 3 strand diameters in case of pre-tensioning. Mix design of concrete for PC deck has to respect a 0.45 water to concrete ratio, a S4 ÷ S5 concrete consistency class of at least 45 MPa characteristic cubic strength. A quality assurance system and testing before and during every casting operation reveals the quality of mix design, which is recognised as an important factor for life and durability of PC structures.

3.1.2 *Bearing and expansion joints*

Under simply supported railway bridges, only one kind of bearing is generally present: spherical bearings with polished stainless steel and PTFE plate.

With this kind of bearing, rotations can occur till ±0.0167 rad in all directions, in order to place the bearing without inserting packings. In order to avoid parasite forces arising with only one train on a double line bridge deck, a new kind of fixed bearings has been studied: it has a special device which controls horizontal stiffness.

The expansion joints are realised with dielectrical elastomeric cushion joints, composed by neoprene reinforced with vulcanized steel plates. They allow fast bearings changing with a maximum differential lifting of 50 mm between decks, operated by hydraulic jacks between deck and pier cap (all simply supported or continuous decks have to pass this design verification) without any operation under rails. Actually, the use of mechanical devices instead of resins avoids disease to daily train in case this operation becomes necessary, or not enough space to insert hydraulic jacks on pier caps, etc.

On pier caps and abutments of every bridge span, reinforced concrete or steel devices ("stroke end device") are required in order to avoid deck slipping out of pier cap and falling, because of accidental breaking of fixed bearings e.g. in case of devastating earthquakes. There are pillows of reinforced neoprene where decks may hurt against these provisions and their maintenance or changing operations has to be assured by proper design.

Italian standard for railway bridge bearings and expansion joints requires these devices underpass preliminary homologation tests led by F.S. technicians, through prototypes testing, in order to assure quality of every single component and of the final assembled products.

3.1.3 *Piers and foundations*

Piers have usually circular or rectangular, full or empty, cross-sections, while foundations are usually realised with plinths with large diameter reinforced concrete piles.

In case of piers in riverbed, even if empty structural sections are adopted, low class of concrete is always poured inside till the river maximum level, in order to avoid unexpected water inside.

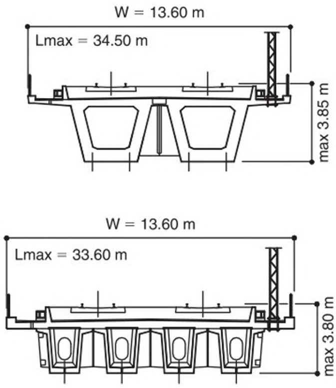
As previously mentioned, in order to increase structural safety, all bridges are designed considering at least low seismic condition: it focuses the designers' attention especially on reinforcement details, very important for piers and piles. Good number of stirrups and loops for longitudinal bars and concrete confinement, use of hooks for good stirrup behaviour, limitation of maximum compression stress in pier concrete, no junction or superposition of longitudinal bars in the length of 3 m from foundation, etc. are consequences of above-mentioned prescriptions.

The minimum reinforcement areas for both piles and piers is fixed to the 0.6% area of concrete section, and spirals are admitted as stirrups in reinforced concrete piles only if welded to longitudinal bars in every intersection.

3.2 *Decks' typical cross sections*

The most common cross-sections of prestressed concrete decks are showed in Figures 2, 6 and 11; in the following, a brief description of main features is presented for each typical cross-section.

Type "a" is a box girders deck, spanning till 34.5 m, generally composed by two precast box girders, prestressed with longitudinal steel strands and connected with small second step casting in



Type **a**

Weight of one precast box: 455 ton (33.1 m)
19% of total length of viaducts
915 ton one deck weight (34.5 m)
343.196 ton is total weight

Type **b**

Weight of single precast V beam: 88 ton
40.5% of total length of viaducts
650 ton one deck weight (25 m)
698.476 ton is total weight

Figure 2. Two box girders deck (a) and four precast V beams and cast in situ slab (b).

the slab and with transversal beams with post-tensioning cables. In Roma-Napoli line it was also realised with two V-beams and cast in situ slab. Transversal beams are usually prestressed with straight cables of strands or bars.

The number of transversal beams is prescribed in [1]: for a deck with two or more girders, at least two prestressed concrete transversal beams have to be designed out from supports and more in case of decks longer than 25 m.

Strands getting out from the heads of the box girder are cut, isolated and protected with the use of dielectric resin. Decks' deformability is largely verified for comfort limit state: maximum deflection at midspan for Type "a" is less than l/5600.

Type "b" is composed by four precast V-beams and cast in situ slab: actual maximum length is 33.6 m.

Beams are steam cured, pre-tensioned with longitudinal steel strands and transversally connected with cables in transversal concrete beams; it is the most common deck: it has been chosen for 40.5% of total length of simply supported prestressed concrete deck.

The maximum deflection at midspan is largely verified: for type "b" spanning 25 m (22.3% of length of all prestressed concrete decks) it is less than l/6000.

Because of the prescription of complete absence of tensile stress during construction and life of the bridge, and of the large amount of pre-tensioning strands in V-beams, the technique of strand passivation for few metres along beam ends, over supports, has been introduced for a portion of strands, in order to reduce even minimal cracking on heads of the beam.

From an aesthetic point of view, shortest spans of box girders (both V-beam or cellular deck) can be put at a disadvantage, because in [1] a free height of at least 1.6–1.8 m inside box girder is prescribed to be left to ease inspection, leading to relevant height of the deck even for short span bridge. Anyway, these spans can be agreeably inserted in case of viaducts with short piers.

Type "c" is a single box girder deck, realised in two different ways and lengths: 25 m long precast box girder with longitudinal pre-tensioned steel strands on Torino-Milano line (3.78 km long Santhià and 1 km long Carisio viaducts) and cast in situ post-tensioned deck spanning 43.2 m (2.8 km long Padulicella viaduct) on Roma-Napoli line.

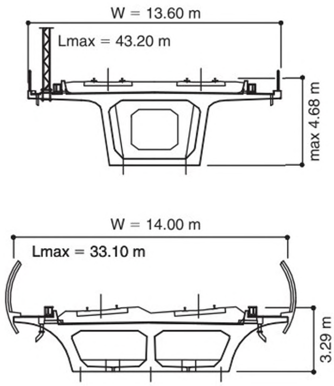
Type "d" is adopted in 5.1 km long Piacenza viaduct: 150 precast-spans with two cells and curved transversal profiles. It is a single monolithic box girder of 970 ton, with a maximum length of 33.1 m.

Piacenza viaduct has been provided with 119 km of corrugated plastic ducts and it represents the first application of electrically isolated disposals for the anchorages of 12 and 19 strands for longitudinal post-tensioning cables. Compared to the grillage decks, the monolithic decks have the aesthetic advantage of clean prospects and even deck sides, without transversal beams or second

Figure 3. Prestressed concrete V beam (type b) in stocking area, Torino – Milano line.

Figure 4. Box girder (type a) on carriers towards launching operations, Milano – Bologna line.

Figure 5. The first precast 25 m long single box girder (Torino – Milano), during launching operations.



Type **c**

Weight of single precast deck: 567 ton (25 m)
Weight of cast in situ box deck: 1043 ton (43.2 m)
11% of total length of viaducts
173.856 ton is total weight

Type **d**

Weight of single precast deck: 970 ton
7.2% of total length of viaducts
145.500 ton is total weight

Figure 6. Box girder with single cell (c) and Box girder with two cells (d).

step castings of concrete in head anchorages of tendons in the required transversal beams, which always become visible with time. Anyway, in some case of grillage deck, when perspective had to be improved, concrete noise barriers have been usefully adopted, covering second step casting or empty spaces on pier-cap between decks for inspection.

Type "e" is composed by four precast I beams and cast in situ slab.

Beams are longitudinal post-tensioned with cables composed by strands with straight and parabolic profiles. Some are tensioned in precasting plant, then, after completing the bottom slab and tensioning of transversal cables, the second part of longitudinal cables is tensioned over the piers and slab is casted. Longest span of type "e" is also the longest span for simply supported prestressed concrete decks: 46.2 m.

Type "e" has the advantage to manage precasting and launching of one beam at time instead of full deck, so requiring simpler technology, but, at the same time, the operation of assembling formworks and casting connection of lower slab and transversal beams, and the huge transversal post-tensioning (no. 47 4-strands cables for 46.2 m long deck) may put it to a disadvantage.

Type "f" is the original Modena viaduct: the first case of lower way U deck for high-speed lines; the double track is realized by two independent decks and piers and common foundation. It has two single track decks spanning 31.5 m, and a total width of 18.4 m; each deck is pre-stressed with 20 longitudinal post-tensioning tendons of 12 strands. 566 km of corrugated plastic ducts is used. In Milano-Bologna line this structural solution for prestressed concrete deck is used for 10.7 km long double track viaducts. It is also used in Junctions, for another 3.33 km of single-track, for a total number of 767 precast spans. In particular, Modena viaduct is the longest viaduct of all high-speed lines, with its length of 7.1 km.

The structural solution of lower way deck has been usefully adopted to minimize structural height plus noise barrier, because the noise barriers became a part of the structure. Modena system of viaducts is one of the few cases, where aesthetics and environment impact have so strongly led the process of design and the solutions for structural and construction needs, in order to have quite original deck, unique in its kind.

3.3 *Comparison between a–f typologies*

Last data about PC simply supported spans are in Figures 14 and 15 where the most important figures about double-track decks of the new high-speed lines are presented.

There's good uniformity between different typologies dealing with deck load and reinforcement and prestressing steel amounts for each span, apart from few exceptions.

Talking about deck load, first exception to be mentioned is Modena deck: as a single way deck it results heavy solution for a double line, but, at the same time, the simple "U" profile, easy to

Figure 7. Piacenza viaduct precast deck, 33.1 m, in stocking area.

Figure 8. Perspective of curved profiles of Piacenza viaduct.

Figure 9. Four precast I beams in stocking area, from Roma-Napoli high-speed line.

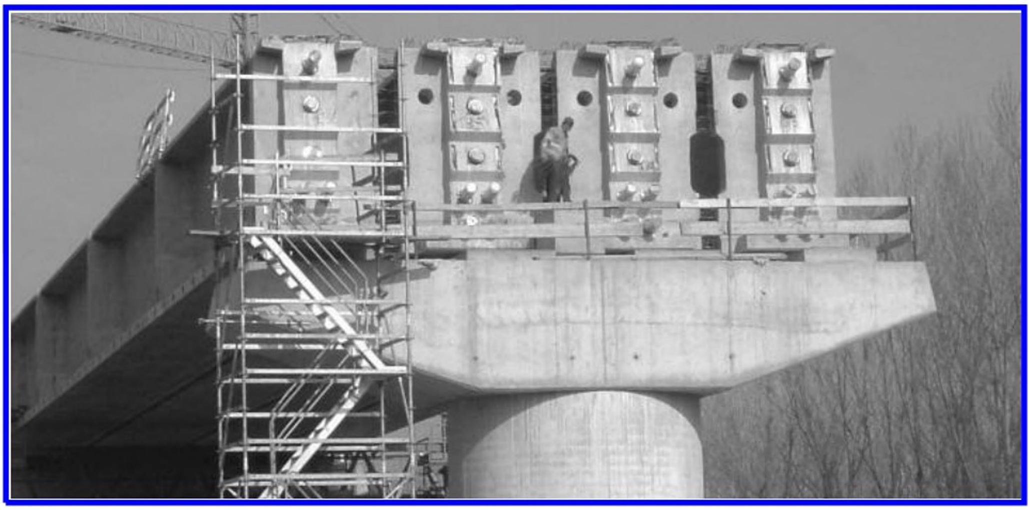
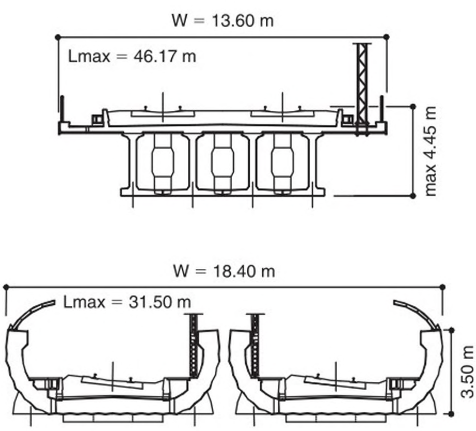


Figure 10. Four precast I beams launched over pier caps, from Milano-Bologna high-speed line.



Type **e**

Weight of single precast I beam: 270 ton (46.2 m)
Weight of four I beams deck: 1.400 ton (46.2 m)
7.6% of total length of viaducts
137.274 ton is total weight

Type **f**

Weight of single precast deck: 689 ton
15% of total length of viaducts
446.472 ton is total weight

Figure 11. Four precast I beams and cast in situ slab (e) and Lower way U deck (f).

Figure 12. The first one of 750 Modena precast decks in stocking area, Milano-Bologna line.

Figure 13. Beam head of Modena precast deck in stocking area, Milano-Bologna line.

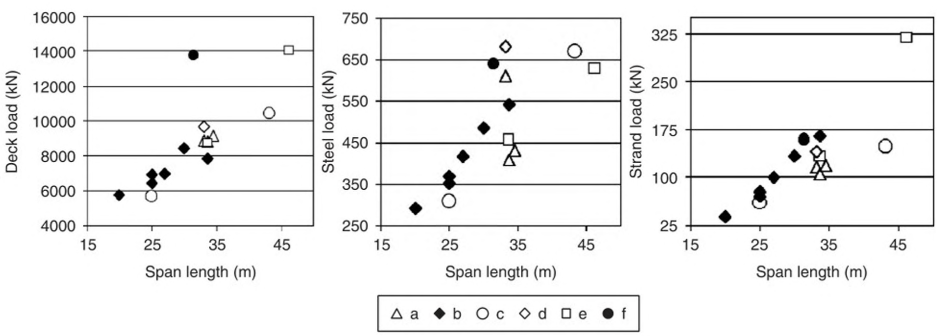


Figure 14. Concrete, Reinforcement and Strands loads of different typologies of deck versus span length, (figures based on high-speed lines: Roma-Napoli; Bologna-Firenze; Milano-Bologna; Torino-Milano).

manage from a design point of view, doesn't cause similar examples of exception for the load of reinforcement and prestressing steel. Decks composed by beams can never minimize the use of steel because of their transversal connections, while single box girders seem, even if based on few examples, to behave more efficiently. Anyway, many other factors are to be considered in the choice of the best structural and technological solution for a new railway bridge deck: they may depend on construction method, workmanship factors, number of spans to be built and required time scheduling of construction, as previously mentioned.

The "cost" of a solution for a bridge viaduct is the sum of all these factors, each time re-considered in the particular economical, skills, managing, furniture and technical "environment". All these factors together have led to the span length and typology distribution in the last graph (Fig. 15): it classifies the nearly 2300 spans with respect to their lengths: the most common ones are short decks spanning 25 and about 33 meters.

The most common solution for prestressed concrete deck is the four precast V beams and cast in situ slab, chosen in 40.5% of cases, due to the relatively simple technology and the quoted flexibility. The need of simplifying and speeding up the construction process has led to prefer classical V, I, and T beam profiles for new precast girders. The third position covered by the lower way U deck

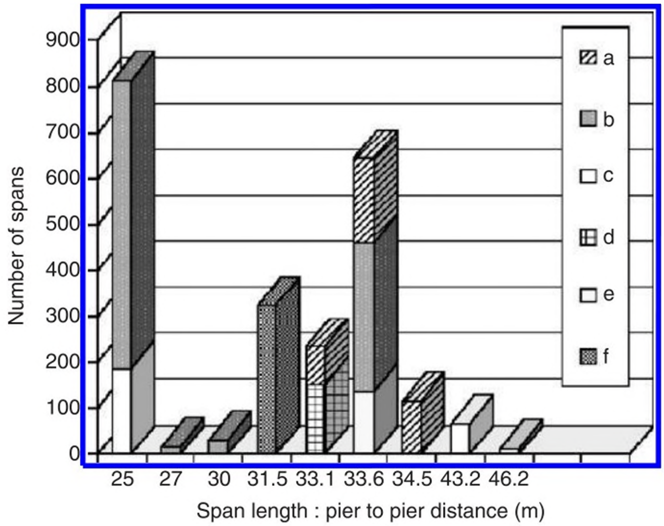


Figure 15. Span length distribution (e) among the a–f typologies (figures based on high-speed lines: Roma-Napoli; Bologna-Firenze; Milano-Bologna; Torino-Milano).

Figure 16. Savena arch bridge along Bologna-Firenze high-speed line during construction.

(15% of total length of viaducts) is mostly due to the relevant length of the unique System of viaducts around Modena, rather than to an actual diffusion of this solution along the high-speed lines. Another factor for evaluating the solution for a bridge deck is, of course, its aesthetic impact: even if mentioned at the end of the analysis, in some case it has strictly led the design choices.

Generally speaking, the span length over deck height ratio can be considered one of the simplest measure to evaluate the grade of slenderness: in case of simply supported PC decks, it ranges between 9 and 12; even for short spans, the prescription of minimum free height inside cellular decks causes small ratio.

Anyway considering that the average piers' height is not more than $7 \div 8$ m, usually long spans are not used with very short piers because of aesthetic reasons too.

Monolithic decks are to be preferred to precast beams decks because transversal beams are always impacting on the sides' prospects.

Finally, a good example of agreeable bridge aesthetics in the field of simply supported spans was obtained changing the structural solution: the Savena arch bridge on Bologna–Firenze high-speed line, designed to overpass Savena River. It is composed by reinforced concrete arch, steel hangers connected by spherical hinges to the 2930 ton prestressed concrete deck, with both transversal and longitudinal post-tensioning cables. It has the lowest height under rails (1.60 m) for a span length of 62.5 m.

4 DURABILITY

Through periodical inspection of old railway lines and particularly of the Roma-Firenze railway line, an analysis of e most common defects on bridge deck have been made in order to work out new guidelines of design and construction of the high-speed lines. Inadequate access for inspection, leaking of waterproofing system of the expansion joints between decks, insufficient cover, not efficient bearings towards maintenance operations as lifting of decks in order to change bearings, or reduced space to insert hydraulic jacks on pier-caps, etc. have been found as most frequent defects.

These circumstances led to the introduction of new or more detailed prescriptions in the Italian standard for railway bridges [1], in order to guarantee better behaviour of structures with time, care for durability of concrete structures, tensioning cables, system of waterproofing, devices for good drainage and anything else whose purpose is to ensure the overall long-term integrity of bridge structure. But, first of all, great care is put to inspection ways to check the conditions of the structures and medium-life elements.

4.1 *Access for inspection*

All bridges are designed assuring access for inspection, testing, maintenance and possible replacement of medium life elements. It must be always possible to walk over bridge decks because a width of min. 50 cm on both sides is left for maintenance people. Inside cellular deck or closed box girders, as previously mentioned, a minimum height of 1.6–1.8 m must be always guaranteed; fixed stairs from deck to pier cap must be provided every 3 spans or 100 m and from piers to the ground every 500 m, for viaducts longer than 1000 m. Over pier cap it must be always possible to pass from one deck to the following and stairs or landings are fixed to ease the movements of maintenance people.

Finally, it must be also possible to inspect bearings and stroke end devices or to operate in case of replacement of bearings or neoprene pillows, so a free height of 40 cm is left between lower side deck and top of pier-cap.

4.2 *Drainage*

It is a key issue about durability; deck slabs are provided with provisions for good drainage and great attention is put to design, testing and layout of all devices. Drainage of expansion joints is assured by a flashing tray of elastomeric material stuck with resins to slabs' ends, in order to avoid leaking over piers and to drain water out of deck sides. Over bridge deck, in order to protect from atmospheric agents, a thick layer of waterproofing is extended, also beneath the footways. In case of sensible pre-stressing system, pre-stressing strands or post-tensioning cable system just beneath deck slab or critical drainage system (types "a", "c" "d" and "f"), a sprayed polyurethane waterproofing of 3 to 5 mm is extended. Checks are been carried out on site for adhesion and thickness by F.S. technicians. This surface treatment has proven to be very long life cycle performant.

In post-tensioned concrete structures, deck anchorages are to be avoided on deck slab; anyway, for all anchorages, design has to avoid leakage to get access to anchorages, providing protection against leaking expansion joints, as water drips.

4.3 *Post-tensioning tendons*

The grouting of the sheaths of prestressed concrete bridges is always done with vacuum technique with a depression of 0.2 bar during injection. It is standard for prestressed concrete deck with post-tensioning tendons because grouting has been recognised as a key operation to ensure durability of prestressing. Usually, it is difficult to ensure complete filling of the ducts: pathology teaches us that the prestressing tendons are more vulnerable in the case of post-tensioning than in the case of pre-tensioning.

Vacuum injection should avoid having air bubbles near high points of tendon profile and in case of simply supported structures high points coincide always with anchorage locations. Besides, all end caps are filled with grout and surrounded by concrete held in place by reinforcement, with no-shrinking concrete and the same compressive strength as deck concrete.

Latest tendencies about durability of anchorages are High Density Polyethylene ducts with plastic end cap left in concrete and the use of electrical isolated tendons as a further protection of the tendon and mean for monitoring: in Milano-Bologna and Torino-Milano railway lines, a large scale application of plastic ducts and electrical isolated tendons has been undertaken. Italferr has taken the Technical Report from fib (fib 2000) and Swiss guidelines on electric isolated tendons (2001) as standards, asking for mechanical and chemical tests, field measurements to be undergone; a System Approval testing has to be conducted on site on every first application of a prestressing system.

4.4 *Deck equipment*

Every bridge deck is furnished of railings on both sides, anchorages of noise barriers for their future assembly, electric traction poles, stairs to pier cap and from piers to the ground. Great attention has been put to deck equipment: every steel finishing is installed with linkages electrically isolated from deck reinforcement and connected to a dissipative end in the earth for safety reasons. Stainless steel is preferred for the most sensible connections.

4.5 *Electrical isolation of structures*

Every bridge deck is electrically isolated through isolated bearings and expansion joints, from piers and the other decks, besides, in order to prevent and protect bridge reinforcement against strain currents, few disposal for every deck are disposed in an accessible area in order to measure strain current and isolation grade after traffic activation.

This is probably a minor problem on high-speed line decks, but felt deeply in every normal bridge deck and in the Junctions of high-speed line. Whenever problems of potential differences should arise, structures will be electrically connected to earth or a cathodic protection should be eventually adopted.

In case of post-tensioned structures, all anchorages are electrically connected (when no electrical isolated tendons is adopted) and the terminal is drawn out of the structure in order to provide eventually in the future the same provisions as for deck reinforcement, otherwise, in case of pre-tensioned decks, the head faces of the beam are protected with synthetic dielectrical resins.

4.6 *Monitoring and maintenance*

In order to improve deck's behaviour knowledge and control it with time under the influence of external agents (environmental actions, traffic loads, seismic events or exceptional hydrogeological events), a complex monitoring system integrated with the high-speed line has been designed.

At least one section (deck, pier, foundation, piles) per viaduct and, in case of long viaducts, one section every 1000 m is instrumented: it means a large number of strain-gages, inclinometers, thermocouples, instrumented bearings, load cells, foundation settlement meters, piezometers etc. Seldom accelerometers are provided in order to evaluate dynamic response of the structures also in case of seismic actions.

Maintenance program of high-speed lines is essentially based on maintenance actions followings inspection visits: in the code 44/c of Italian railways about lines maintenance [5], frequencies, ways of inspection and following check schedules are prescribed. These check schedules have the double aim to check the safety of structures towards train traffic, and to keep memory of time evolution of the behaviour of structures. According to [5] every year a program of action has to be adopted to eliminate anomalies encountered in structures or to face critical situations.

5 MODELS AND TESTING

A useful key to reach good results since the first casting or tensioning of new structural solutions has been found to be all preliminary tests, both on site and in laboratory.

Mix design of concrete and the phases of casting to be applied with prestressed beam with dense reinforcement cage or post-tensioning cables very close to each other, or designed to be very early prestressed, is always tested on full scale portion of beam, before the beginning of production. In some cases, as Modena viaduct, deck formwork profiles and movements have been deeply studied with some portions of full scale deck to be extracted from formwork, in order to guarantee the result and protect the curves of the section. In other cases, as Piacenza viaduct with five 19-strands very close in vertical line on a web of $50 \div 60$ cm, both frettage reinforcement and minimum concrete

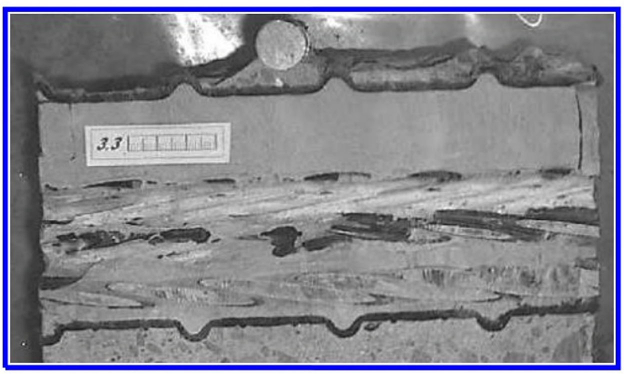


Figure 17. Core from System Approval test.

Figure 18. Electric resistance measurements.

resistance at stressing have been deeply investigated in order to avoid concrete cracking on real decks.

Tensioning of pre-stressed strands is checked on the first presetressed concrete beams with load cells, in order to investigate the decrease of tension after cutting of strands on beam-ends.

In case of post-tensioning, ducts' assembling, tensioning and grouting with vacuum technique are fully tested before the first deck casting on small portion of structure; at the end of testing cores are extracted and examined for system acceptance.

As previously mentioned, all details of deck equipment, waterproofing, linkages are studied and tested before being accepted and applied. Also their layout on decks is tested during construction.

Besides, the first pre-cast elements of every new pre-casting plant must undergo a loading test: load corresponds to relevant percentage of maximum design load (e.g. load corresponding to 96% of concrete cracking moment M1.2*f_{ctk} at midspan section). In those cases of full-span pre-casting, when it can't be reached with normal settlement test, as the load on placed girders is comparable with design load, testing loads are reached with the same pre-cast beams, obtaining fast and low-cost test settlement.

Apart from preliminary test on models and final load tests on every bridge and viaducts, another kind of test is planned at the end of works and before the beginning of working of every high speed line; it is a new system to check in an extensive way all the main structures present: a very special train (extremely heavy compared to normal ones) has been built to achieve the theoretical Load Model adopted in the structural design according to Eurocode and [1].

This special train applies maximum design load on all girders, check structural behaviour of every span and all deformability parameters of bridges and viaducts of the new high speed lines are registered for, at least, 20% of spans.

6 CONSTRUCTION PROCESS

As all bridge designers know, construction process may deeply influence design choices, also in case of pre-casting prestressed concrete beams, which compose the majority of our new bridge and viaducts: so the time steps, the phases, the methodologies of lifting, transporting and lowering over the piers are fully investigated.

To improve the overall quality of the infrastructure design and production, a quality control system was implemented during both the design phase and during the construction phase. As showed in Figure 12, to speed-up the construction there's a big tendency towards precasting.

In the following, main construction features of each structural solution are described in the text and representative construction phases are showed in the pictures.

6.1 *Precast beams and cast in situ slab*

When talking about construction features we must divide our decks in few major families: one of these is the precast V or I beams with cast in situ slab. The short spans of four V-beams deck of type "b" and I-beams deck of type "e", of an average load of 100 tons each, are the only ones which can be casted and pre-tensioned in pre-casting plant, moved on ordinary roads and led on site where each beam is lifted to its final position, over pier-caps. Then predalles or formwork of the slab is assembled, reinforcement is laid and the concrete slab is casted over the piers.

As it is not necessary to pass over completed decks, ordinary roads can be used and there's no obliged sequence of spans' layout, this method of construction has the important property of flexibility; besides, relatively simple technology is necessary for its realization.

6.2 *Cast in situ box girder*

In the only case of single box girder Padulicella viaduct, the deck is casted and pretensioned over the piers on self-launching formworks and special casting equipment is used. To accelerate

Figure 19. Precast V beam lifted up from stoking area.

Figure 20. Precast V beam towards final position over the piers.

Figure 21. Pre-assembling of reinforcement cage of Padulicella viaduct, Roma-Napoli.

Figure 22. Reinforcement cage transported in the formwork over the piers, Padulicella viaduct, Roma-Napoli.

Figure 23. Launching operations of a box girder of S. Rocco al Porto 2 viaduct, on Milano-Bologna line.

the production, the reinforcement cage had to be pre-assembled, transported and lowered in the formwork before casting.

6.3 *Two box girders*

The two precast box girders, each is about 450 tons, are precast and prestressed with pre-tensioned strands, then lifted over the viaduct where they are transported by two small carriers on tyres, towards launching operations.

Deck is completed over piers, with second step casting in central slab and in transversal beams, then transversal strands cables are tensioned and grouted.

The case of two precast box girders is half way between the precast beams and the full-span pre-casting: actually launching operations similar to those of full span pre-casting are necessary, but second step casting and transversal prestessing in transversal beams are needed.

As the full span precast decks of the following chapter, these box girders are realized in a plant near the viaduct: all these plants are dismantled at the end of the works.

Figure 24. Modena precast deck lifted and transported on tyres from stocking area, Milano-Bologna line.

Figure 25. Piacenza precast transported on steel wheels from stocking area towards launching, Milano-Bologna line.

6.4 *Full span pre-casting*

In those cases when much more spans have to be built in little time, full span pre-casting has been preferred.

According to this process of construction, decks are totally pre-casted; no post-tensioning or transversal beam is needed to complete the deck over piers. Afterwards, one by one they are moved towards launching operations.

A «carrier» and a «support beam» always compose the launching system. The carrier slowly moves on tyres or steel wheels, lifts a stored beam, and transports it along the viaduct, moving on the placed girders. The device for transport forms an integral part of the device for the launching of the girders: the carrier drives then into a second steel girder called support beam, suspends the beam over its final position. The support beam is drawn back and the beam is lowered. With four or six bearings hydraulic jacks or load cells are used in order to check weight load distribution, then the beam is lowered on its final bearings.

Figure 26. Modena precast lower way U deck during launching operations, on Milano-Bologna line.

Figure 27. Santhià precast single box deck during launching operations, Torino Milano line.

The use of pre-assembled reinforcement cages, independent casting lines and several formworks, different phases of tensioning operations and the use of storage areas for cables' injection can speed up the process.

On the other hand, if for any reason one deck has to be stopped in the stocking area, the process may stop for days. So this method doesn't have the flexibility pointed out before for beams and slab decks.

Peak cycle is variable: Modena viaduct has a casting and launching speed of two precast girders per day. Others, as Piacenza viaduct, have the design of the spans and of the casting yard centred around a target peak cycle of two double-track deck beams per week.

Figure 28. Piacenza precast box girder with two cells during launching operations, on Milano-Bologna line.

7 CONTINUOUS PRESTRESSED CONCRETE BRIDGES

In few cases, for riverbed or embankment crossings, highway or railway over-flyings, multi-span continuous PC bridges have been designed. The most common choice for a single long span in a sequence of shorter simply supported spans is the composite steel and concrete deck, from about 40 m to more than 70 m, but they result often strongly impacting with longer PC viaducts perspective, consisting generally of a single exception, with different structural height, colour and side profile.

In other case, the need to harmonize approach viaduct spans length to a bigger structure as arch bridges or cable-stayed bridge has led to multi-span PC viaduct with spans of 60 ÷ 70 m.

According to ref. [1], all previously mentioned design and durability prescriptions have to be applied to continuous bridge: access for inspection of every pier cap is more stringent and difficult to obtain, leading often to complex systems of stairs and landings around central supports.

Besides, every deck has to be verified for lifting in case of bearings' replacement and it may result structurally demanding, while, from a technological point of view, it can require a specific design to give disposals for 20% additional prestressing in each span longer than 40 m.

PC continuous beams are often casted in phases, it is quite rare to have a single casting operation for two or three spans and in [1] precast segmental construction is forbidden. For cast in situ segmental PC bridge, a minimum reinforcement area through every joint of 3.0% area of concrete cross section of deck is prescribed and minimum compressive stress of 1.0 MPa (rare load combination, also during construction stages) is expected from design.

In the following, because of lack of space, only two examples of continuous PC bridges are mentioned.

The first example is composed by the river embankments approach viaducts to the cable stayed bridge over Po River [2]: five spans on the left side for a total length of 260 m (Fig. 29) and three spans on the right side for a total length of 130 m. The decks are three cells box girders, built by balanced cantilevers from central piers: a couple of segments at one time is casted in situ and post tensioning cables are tensioned.

Then the remaining gaps of 1.0 m long are concreted and post-tensioning cables are laid in the spans to join together, two by two, the cantilevers. For the Italian State Railways, this is the second

Figure 29. First balanced cantilever of Left Embankment viaduct, approaching cable-stayed bridge over Po River, Milano-Bologna line.

Figure 30. Continuous PC beam of Modena viaduct over Brennero highway, Milano-Bologna line.

example of this kind of structure and method of construction (the first one being built for the Rome-Firenze line).

The second example refers to the Modena continuous bridges to overpass two rivers, a highway and a railway Junction in the Modena System of Viaducts, for a total number of nine single-track continuous beams.

All nine bridges have span lengths of 40–56–40 m, the same outer cross-section of the lower way U decks of Modena simply supported spans (type f), with higher webs on central piers. They are built in three segments casted in situ e prestressed with 40 mm bars, coupled at the end of each segment.

Two methods of construction were experimented for the same structural solution, because of the different environmental conditions: for two of these beams, the segments were casted on a formwork, scaffolding from the ground with two temporary bars for each joint, in order to help the partial structure during construction conditions. The other seven obtained the same effect with a formwork hanging from steel box beams (over Panaro River) or truss beams (over Secchia River and on Brennero highway, see Fig. 30) and the end support of the beam applies nearly the same reaction of the temporary bars.

8 CONCLUSIONS

This paper has described the strategic choices and the most relevant structural and infrastructural features for the design and construction of the new Italian high speed network. Main topics concerning simply supported prestressed concrete decks for new high speed lines have been analysed.

All lessons learned with the experience gained with old railway lines, and especially with the Roma – Firenze line, built in the 70s–80s, together with new technologies of the last ten years, have been applied to improve quality and durability of all new prestressed concrete structures, as all other types of structures.

Deck with precast beams and cast in situ slab is the most common choice due to its flexibility (type "b" is actually the most used structure for bridge decks) but full span pre-casting is the most important future trend.

The mentioned principles of design, the so-called multi layer protection system (as good drainage, easy access for maintenance, durability of tendons etc.), the method of construction, which deeply influences design choices, and the tendency to experiment models and tests before every first realization, have been recognised as strategic factors for good results in design, construction and management of railway infrastructures.

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CHAPTER 6

Bridges for the high speed railway lines in Spain. Design criteria and case studies

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ABSTRACT: This article synthesises the design criteria of the new bridges designed for the High-Speed Railway (HSR) Lines in Spain, remarking the most relevant aspects. The contribution reports several case studies, illustrating the structural behaviour and construction procedure, based on the experience of the author in the design of more than 13.5 km of HSR bridges, including the two first steel viaducts on the line Madrid-Barcelona-French Border.

Keywords: *High-Speed, Railway, Bridge, Design, Construction, Concrete, Composite.*

1 INTRODUCTION

After the conclusion of the first Spanish High Speed Railway Line in 1992, connecting Madrid and Seville, the strategic railway infrastructure plan developed by the Ministry of Public Works as a part of the objective of the European Union (EU) of developing a Trans-European High-Speed Rail System, includes the construction of more than 4000 km of HSR in a period of fifteen years (Figure 1) [1]. Rail represents about 120 billion euros of expenditure under the Plan. The directives for the rail system interoperability constitute an impelling element for the railway sector, as new lines, trains and equipment within the EU countries should be either built or renovated.

As a result of the complexity of the Spanish geography, approximately a 10% of the railway system consists of bridges and tunnels.

Almost all the bridges on the Spanish HSR network are made of concrete, in general built 'in situ'. The first composite steel-concrete bridge built for the HSR in Spain, a box girder with a length of 1.2 km with a main span of 60 m, has been recently been completed (2005) for the Córdoba-Málaga corridor [2]. Two new steel bridges for the Madrid-Barcelona-Perpignan line are currently under construction and expected to be completed by the end of 2006. The first one is located in Llinars de Vallés, crossing at a very skew angle the AP-7 highway, about 45 km to the north of Barcelona. The second bridge is located in Sant Boi del Llobregat, close to the Barcelona airport.

2 DESIGN CRITERIA

2.1 *Design codes*

The HSR lines in Spain are developed by the Ministry of Public Works. The technical specifications required by the owner for the design of these bridges are as follows:

- Loads should be according to the Spanish Code for Railway Bridges but a check is also required to fulfil the specifications of Eurocode 1.2 (Traffic loads on Bridges) and the Annex 2 of Eurocode 1, specifying additional Serviceability Limit States.
- Design of structural elements should be carried out according to the Spanish Codes for concrete structures or the recommendations for the design of composite and steel road bridges.

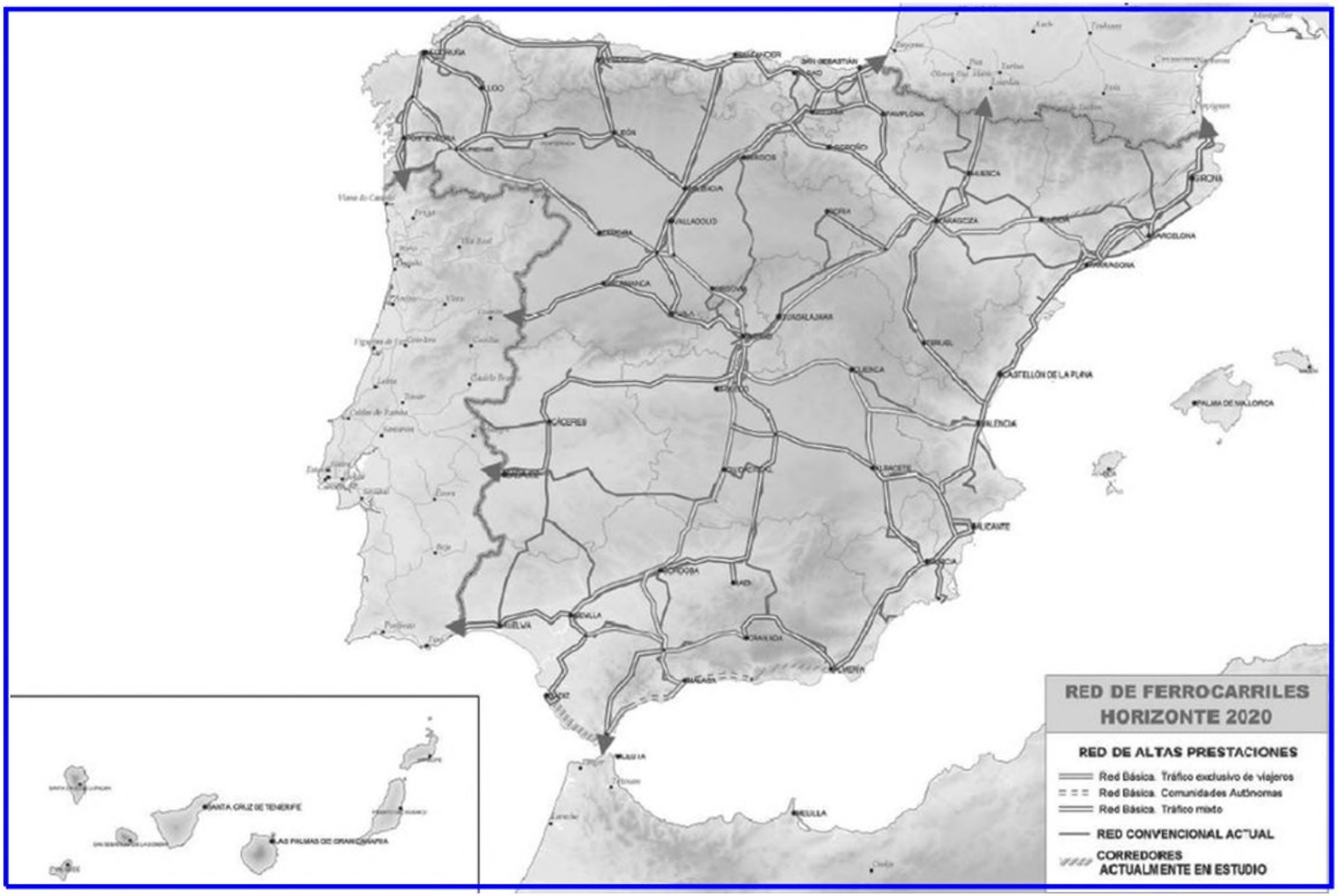


Figure 1. View of the expected Spanish High Speed Rail Network in 2020. Map obtained from [1].

Table 1. Comparison between standard road and railway bridges.

Load (kN/m)	Road bridge	Railway bridge	Railway/Road (%)
Self Weight (D1)	175	280	160
Dead load (D2)	45	180	400
Equivalent Uniform Live load (Q)	60	200	333
TOTAL D + Q	280	660	235

2.2 *Specific relevant aspects for the design of HSR bridges*

Internal forces due to railway traffic loads are 2 to 2.5 times larger than those induced by road traffic loads. Dead loads of two ballasted tracks, including all bridge finishes, weights 120 kN/m and the effect of ballast should be incremented for the design about 30% to take into account possible increments during the life of the bridge. Typical values of loads for a standard deck, with a mean span length of 40 m, with two tracks (width of 14 m), are illustrated in Table 1.

Horizontal loads originated by railway traffic (breaking and traction, nosing force, wind, track-structure interaction and centrifugal forces) are also much bigger than similar effects in road bridges. As an example, maximum breaking and traction force in a HSR standard 300 m viaduct is 7000 kN. The equivalent force in a similar road bridge is 850 kN. Loads induced by centrifugal forces in railway bridges could also range from 300% to 1500% of the ones caused by the same force in road bridges.

Apart from the heavy loads considered for the design of HSR bridges, there are some specific Serviceability Limit States (SLS) to be verified in this type of bridges that could be summarized as follows:

- Verification of vibrations for traffic safety, limiting the maximum vertical peak deck acceleration induced by real trains (for instance, the recommended value for a ballasted track is 5 m/s^2).
- Verification of deck twist and vertical deformations of the deck for traffic safety.
- Verification of the maximum vertical deflection for passenger comfort, depending on the train speed and span length.
- Verification of track, limiting rail stresses due to combined response of the structure and track to variable actions, limiting the longitudinal displacements induced by traction and braking (for instance to only 5 mm for welded rails without rail expansion devices or with only one expansion device at one end of the deck).

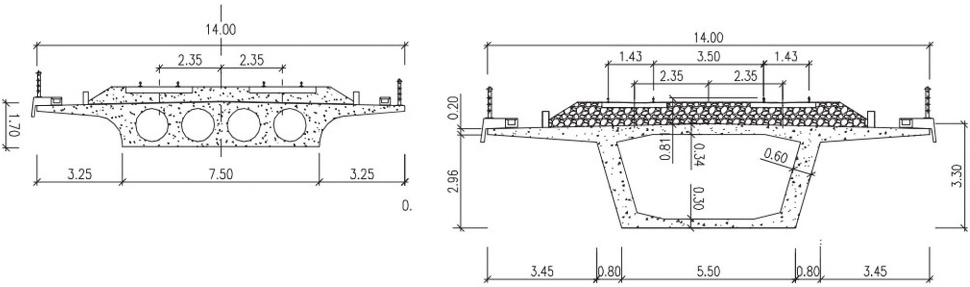
Due to these significant loads and the very strict Serviceability Limit States to be checked, structural elements are much stiffer than in road bridges and for this reason, the optimization of materials and, in particular, the selection of a judicious structural system and the deck's slenderness is basic to obtain economical solutions.

3 CONCRETE BRIDGES

3.1 *Concrete bridges cast in situ*

3.1.1 *Structural types*

An important ratio of the viaducts in the HSRL in Spain presents medium span lengths ranging from 30 to 60 m and most of these bridges are made of cast in situ post-tensioned concrete. The deck of these bridges is 14 m wide, accommodating two ballasted tracks. For spans varying from 20 to 35 m length, the most appropriate typical deck cross-section is a voided slab (Figure 2a) with slenderness (span/depth) ranging from L/15 to L/20. For larger spans the unicellular box girder is



(a) (b)

Figure 2. Typical bridge cross-sections for HSRL Viaducts of medium spans. (a) voided slab (b) box girder.

Table 2. Ratios of materials per deck surface area in prestressed concrete decks cast in situ (mean and coefficient of variation).

	Concrete (m^3/m^2)		Prestressing steel (Kg/m^2)		Reinforced steel (Kg/m^2)	
	Mean	C.O.V.	Mean	C.O.V.	Mean	C.O.V.
Voided slabs	0.83	0.10	25.0	0.13	85	0.12
Box girders	0.80	0.09	27.1	0.13	107	0.08

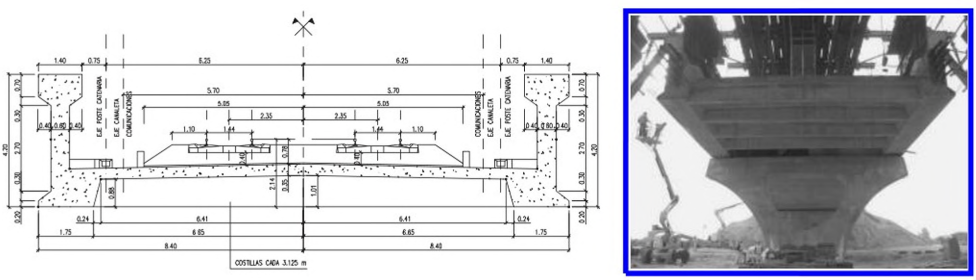


Figure 3. Typical bridge cross-section for a HSRL viaduct with a main span of 50 m.

the common solution (Figure 2b) with slenderness varying between L/12 to L/17, depending on the construction method.

The ratio of materials used in these cast in situ prestressed concrete deck solutions is summarized in Table 2. The ratio of prestressing steel (RPS expressed as kg/m^2) could be estimated in continuous slab bridges by RPS = 0.95*Lmax (m). In case of box girder bridges this ratio could be estimated as RPS = 2.03*(Lmax/depth).

In case of vertical clearance restrictions, other cross-sections may be used as an alternative to those mentioned above (Figure 3). This solution was designed for a continuous bridge of 530 m in length with a maximum span of 50 m (slenderness of L/11.9) and constructed using the span by span method with classical scaffolding. In that case, the ratio of prestressing steel (RPS) has been 38.9 Kg/m^2 of bridge surface area (14 m width).

The ratio of prestressing steel is clearly dependant on the admissible concrete tensile stresses. Most of the concrete bridges included in this paper have been designed to avoid cracking under rare (characteristic) load combinations. Tensile stresses are admitted smaller than characteristic tensile concrete strength-under frequent load combinations. With this criterion loss of stiffness is avoided due to cracking (the dynamic behaviour of the bridge remains elastic) and fatigue effect in

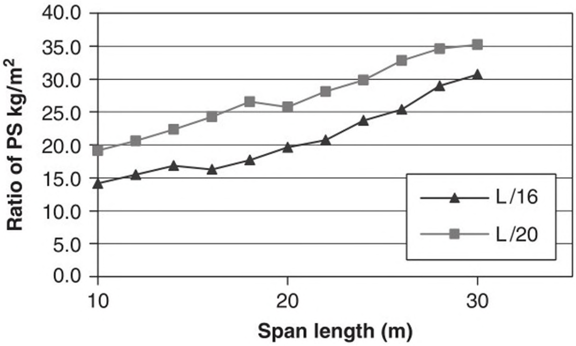


Figure 4. Ratio of prestressing steel in slab bridges for two different slendernesses.

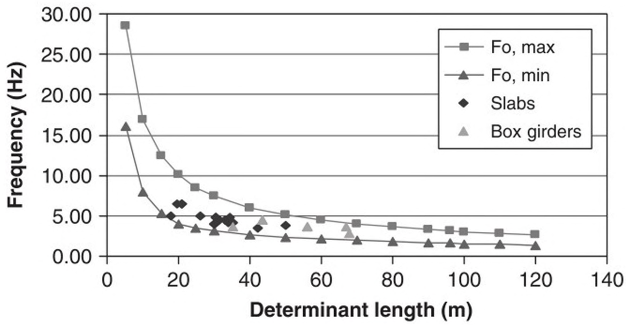


Figure 5. First flexural frequency of slab and box girder bridges and limits of EC-1 for train speeds lower than 200 km/h.

the prestressing steel is practically negligible. Figure 4 shows the ratio of prestressing steel needed in a voided slab type deck for two different slendernesses.

3.1.2 *Structural behaviour*

Dynamic behaviour of all the bridges presented has been analyzed according to the procedures included in Eurocode 1.3 [5,6]. For each structure it has been necessary to study the load effects induced by real trains at different speeds (up to 420 km/h). The maximum acceleration depends on the slenderness and geometrical configurations. As a general conclusion, the impact factor obtained has a mean value of 1.75, but considering that real train loads for high speed lines weights about 25 kN/m the internal forces induced by these trains are smaller than those obtained with the design models proposed by Eurocode 1.2. Maximum accelerations obtained with real trains vary between 0.4 to 1.4 m/s^2 which are smaller than the maximum accepted value to avoid problems with ballast. Figure 5 shows the first flexural frequency of the bridges designed.

3.1.3 *Construction procedures*

Different construction methods have been used for the erection of bridges cast in situ. In general, they have been built using classical stationary falsework or a launching girder, erecting the deck using the span by span method if the deck presents more than 4 spans (Figs. 6 and 7). The incremental launching method is also commonly used in bridges with a length superior to 400 m and with a geometry that permits this type of construction.

Figure 6. Vandellós Bridge. Construction using the span by span method with a total length of 410 m.

Figure 7. Avernó Bridge. Construction using the span by span method with a total length of 810 m.

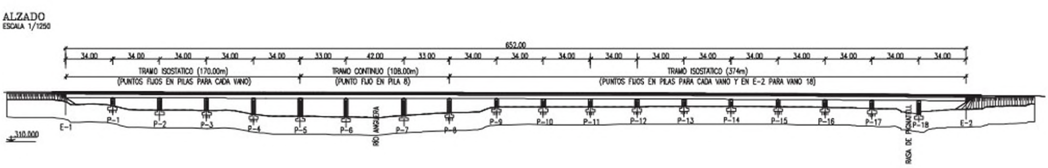


Figure 8. Elevation of the viaduct over Anguera River with a total length of 924 m.

3.2 *Concrete bridges using prefabricated elements*

3.2.1 *Structural configurations*

In case of long viaducts, the use of prefabricated elements could lead to an optimization of cost as a result of highly mechanized and industrialized erection.

This alternative was developed for two similar viaducts using the same prefabricated elements with a total length of 924 m (Figure 8) and 652 m respectively. The bridge deck is constituted by prefabricated pretensioned U-beams with a reinforced concrete top slab, facilitating the erection of the deck with mobile cranes and diminishing the construction time. The typical span is 33 m long, acting as a simple supported beam, except for three continuous spans to cross the river bed with a maximum span of 42 m. In this continuous part of the deck, prefabricated beams have been longitudinally connected using prestressing bars. The typical cross-section consists of two

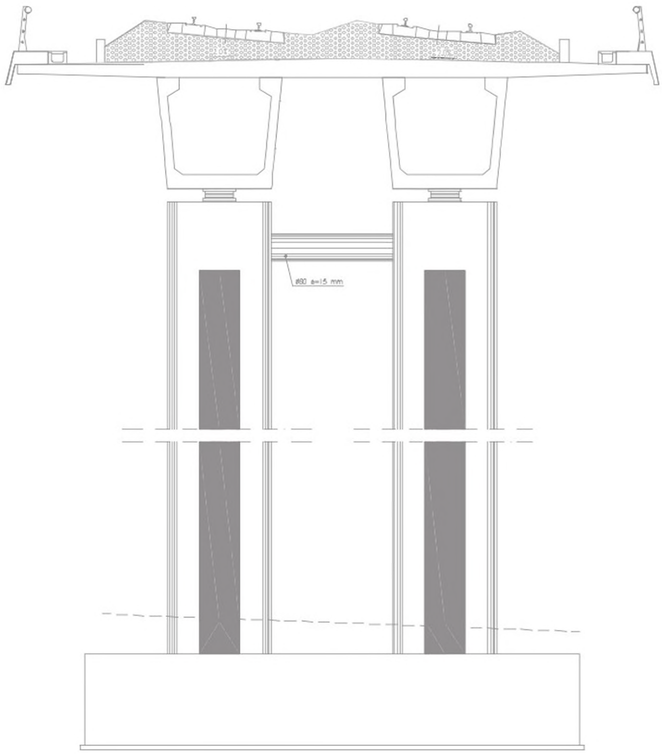


Figure 9. Typical cross-section of Anguera bridge.

Figure 10. Erection of a beam and typical cross-section of the deck.

prestressed concrete U-girders that have been prefabricated at the site – in a temporary purpose built factory – (Fig. 9). The total depth of the deck is 2.8 m (slenderness L/11.7) and the depth of the beams is 2.45 m. The ratio of prestressing steel is 21 Kg/m^2 of bridge surface area. A moving falsework has been used to erect the reinforced concrete top slab of the bridge, with a construction rhythm of 4–5 days per span (Fig. 10). Each span is horizontally fixed at one the ends in the longitudinal direction with a fixed POT bearing.

4 COMPOSITE STEEL-CONCRETE BRIDGES

The two steel bridges in Sant Boi and Llinars have been designed to fulfil similar explicit requirements from the owner (ADIF, Administración de Infraestructuras Ferroviarias), which could be synthesised in the following points:

- The bridges cross existing infrastructures, including very busy motorways and no interruption of road traffic would be permitted neither during construction nor in service.
- Whereas the vertical clearance to the road below was limited to 5.5 m, the structural elements below the tracks had to be kept to a minimum. These requirements have the intention of reducing the elevation of the track to minimise the environmental impact on the surroundings (reducing the height of embankments and limiting the excavations at the rest of the stretch).
- The bridges are located in highly visible sites and they should aesthetically be designed very carefully and fit in well with the surroundings, presenting an image of creative/innovative elements: a symbol of vanguard technology for the owner.

4.1 *General aspects*

To ensure compliance with the specifications of the owner, and after analysing both technically and economically different alternatives comprising concrete and/or steel structures and several structural types including box-girders, I-girders, arches, etc., the final design consist of a composite steel-concrete deck with a maximum depth of 1.45 m below the ballast (2.15 m below the track level) which is suspended from tied curved steel members. The main supported structure is above the deck level and consist of two planes of curved steel suspension members supported from vertical steel pylons (Fig. 11).

4.2 *Design criteria*

Following the owner requirements, the structures have been designed according to Spanish Code for railway bridges [3] for the definition of loads and the Spanish Code for composite steel-concrete road bridges for the dimensioning [4]. Additionally, as required for all Spanish HSR bridges, the design has also been carried out according to the Eurocode 1 [5,6]. Vertical train loads specified by Eurocode are a little bit smaller than those specified by the Spanish Code but the definition of loads for the dynamic analysis are clearly specified as well as track-deck interaction and other specific railway actions. Eurocode 1 is also clearly defining some Serviceability Limit States which are crucial for the design of HSR bridges, in particular the definition of the maximum displacements and rotations at abutments, accelerations, maximum vertical displacements due to traffic loads, etc. Most part of these criteria is similar to requirements of different UIC documents [7,8].

Figure 11. Sant Boi bridge. General view during construction at the assembling area at the site.

As a general conclusions, the Serviceability Limit States and, in particular, the control of deformations, is governing the general design of the steel elements. The bridges are built by incremental launching and, as a consequence, the design of webs and the inferior flange of the longitudinal beams that support the deck are conditioned by patch loading phenomena and to avoid local plastifications. Fatigue has conditioned the design of some details, such as joints between transverse floor beams and longitudinal beams.

The steel grade used in both bridges is S355-J2G3 with different classes of control or limitations of surface defects, depending on the plate thickness and also on its location. For joints or plates that receive welded transverse plates, an exhaustive control with X-Rays has been specified. Concrete used for the deck is grade C-30.

4.3 *Structural behaviour*

Although the geometry of the structures could remind one of a suspension bridge, they behave mostly as a continuous beam with variable depth.

Different numerical models have been developed for the design of both bridges.

- A general linear-elastic model combining beam and shell elements for the concrete deck. An equivalent depth of the concrete shell members has been introduce to take into account cracking in some areas or to consider the different nature of loads (permanent or live loads).
- A general model for the evaluation of construction effects (launching) and for the evaluation of the dynamic behaviour under real trains at different speeds. This model is an elastic model with beam elements. A complete dynamic analysis according to Eurocode-1 has been carried out to confirm behaviour of the bridge.
- Local finite element method models to analyse the distribution of stresses in structural nodes or in areas which are strongly stiffened. The analysis has been made using elastic shell members and, in some cases, considering geometrical imperfections.

As a result of the exhaustive dynamic analysis, following the specifications of Eurocode 1, an excellent dynamic behaviour has been confirmed, obtaining maximum accelerations lower than the admissible limit (0.35 g) for ballasted tracks even for speeds close to 400 km/h (Fig. 12) [9]. Main first flexural frequency for the two structures is 1.28 Hz for Llinars bridge and 1.33 Hz for Sant Boi viaduct, including all permanent loads.

4.4 *Llinars bridge over highway AP-7*

This high speed railway viaduct has an overall length of 574 m. The first part of the bridge is a composite steel-concrete structure crossing the highway; the second part is a continuous prestressed concrete bridge crossing Mogent River with a maximum span of 48 m. The location of

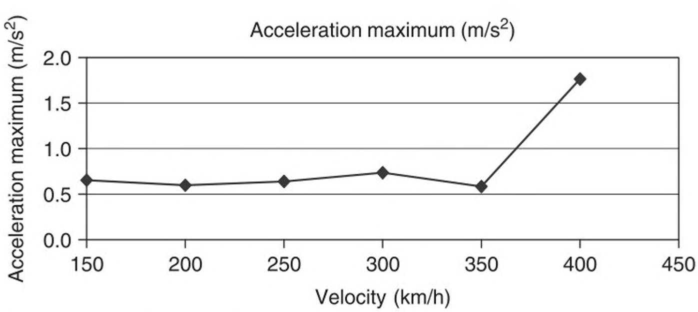


Figure 12. Maximum accelerations due to train n° 3 in EC-1.

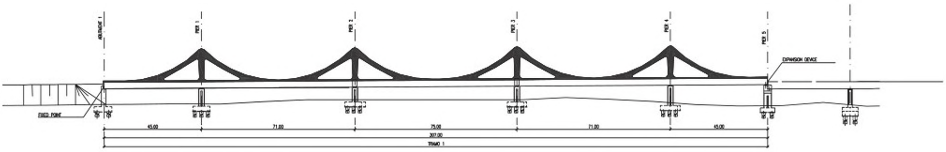


Figure 13. Elevation of the bridge.

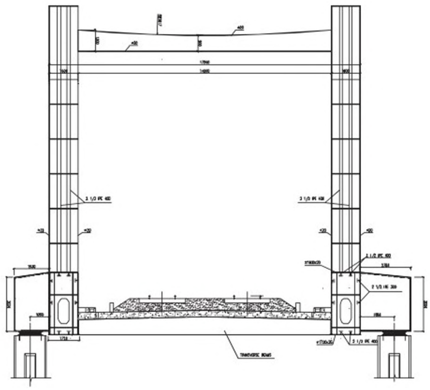


Figure 14. Typical cross-section at the central pier.

piers is controlled by the highly skew angle of the highway crossing and by the erection process (launching). Due to the limited vertical clearance requirement to reduce environmental impact at the site, the depth of the deck should be limited to 2.15 m below the railway track. Nine different alternatives, comprising several structural types including concrete and composite (steel-concrete) solutions were technically and economically studied. The final bridge is a composite steel-concrete deck suspended on structural steel tied members. An effort has been made to develop an aesthetically pleasant solution, very transparent and well fitted to the site. Incremental launching construction method is used to avoid interference with AP-7 highway which is vital for the city of Barcelona.

4.4.1 *Description of the bridge*

The composite part of the viaduct has a total length of 307 m. The deck is a continuous structure with 5 spans of 45 + 71 + 75 + 71 + 45 m (Fig. 13).

The bridge, 17.2 m wide, accommodates 2 ballasted tracks, with a platform width of 14 m (Fig. 14). The composite concrete-steel deck consists of parallel transverse I-beams 1 m deep, 3.55 m apart.

The transverse beams are connected with the 1.6 m wide longitudinal box girders having a varying depth of between 3.5 and 6 m (Fig. 15). The webs are longitudinally stiffened with two ½ IPE 300 or ½ IPE 500 and transversely every 3.5 m or with diaphragms every 7 m. In some areas, centre of spans and below pylons, the space between transverse stiffeners is reduced to 1.75 m. These longitudinal beams are suspended from curved tied steel box members supported from 14.5 m high steel pylons. All the elements are made up of welded stiffened plates.

As piers could only be located at the highway shoulders and at the median strip, some of them are not below the deck. Four piers are separated from the outer part of the deck by up to 4 m. To solve this situation and to have the two pylons in the same vertical plane perpendicular to the deck axis to have a more transparent view of the bridge, a transverse cantilever beam has been designed. This cantilever connects the longitudinal beam with the pier (Fig. 14).

Figure 15. Typical cross-section of the longitudinal beam.

Figures 16 and 17. Views of the suspension members and pylon.

The suspension members also have a typical box-girder cross-section with smooth varying depth with an average value of 1.70 m and with a constant flange of 1.6 m (Figs. 16 and 17). They have been curved with a radius of 48.6 m to improve the aesthetics of the structure, transmitting a quiet vision of this huge bridge with a so limited vertical clearance over the highway.

Pylons (rising 14.5 m above the top chord of the longitudinal beam) are subjected to significant internal forces, in particular, those which are supported by transverse cantilever beams. The cross-section of the pylon is a box girder with a depth varying from 2.1 to 2.7 m stiffened with standard profiles type ½ IPE 400 or ½ IPE 600. The total weight of the structure is 2800 tonnes, which represents an average of 615 kg/m^2 (for a 14 m wide platform).

The coating system to protect the steel members consists of a standard three layer system for the exterior surfaces; after blast-cleaning grade Sa2.5, a first layer of a shop-primer epoxi polyamide

Figure 18. View of colours selected.

Figure 19. View of steel sheet formwork.

pigmented with zinc-phosphate (25 μm), and intermediate layer of epoxi-polyamide (170 μm) and a final layer of acrylic polyurethane (30 μm) have been applied. For interior surfaces the coating system consists of two layers (shop-primer plus epoxi amina 200 μm). Pylons and suspension members, which present difficult access for painting or for future inspections, have been completely sealed. The selection of colour was essential to improve the appearance of this huge structure with a very small vertical clearance over the highway. After a wide analysis it was decided to select a light blue colour for the members above deck level and grey for the bottom part of the longitudinal beams. This combination clearly increases the apparent slenderness of the bridge (Fig. 18).

The structural system for the deck is, conceptually, very clear. It consist of transverse I girders with an average depth of 1.1 m and spaced every 3.5 m. The depth varies from beam to beam because the elevation of the longitudinal beams is straight to facilitate the launching, but the elevation of the track is variable (parabola). With this geometry, the ballast also has a constant depth. The connection between transverse and longitudinal beams has been designed as pinned with a reduced flexural stiffness to avoid fatigue problems at the inner web of the longitudinal beams. The transverse beams directly support an upper concrete slab of 0.35 m depth, which is structurally connected with standard welded shear connectors 19 mm in diameter and 125 mm high (Bernold type). The concrete layer is placed after the steel structure is in its final position, to reduce internal efforts during launching, using as a formwork a cold formed steel sheets (Fig. 19). To use standard profiled sheet of limited height, the pouring of concrete is made in two layers.

The superstructure is supported on concrete piers and abutments and the substructure is founded on piles with a diameter of 1.5 m. As in all HSR bridges, we have defined a fixed point. In this structure, this very stiff element is at one of the abutments.

Figure 20. Suspension member at the steel yard.

Figure 21. Assembling at the site.

4.4.2 *Construction process*

The steel parts of the bridge have been fabricated by URSSA in Vitoria (550 km from the site) and transported in pre-cast units of about 20 m weighting about 50 tonnes (Fig. 20). The bridge is assembled at the site on a platform behind the south abutment (Figs. 21–23).

The bridge is assembled in four segments of about 75 m and with a weight of approximately 700 t (Fig. 20). After the assembly of each segment, that takes about five weeks, the segment is pushed forward to permit the assembly of the succeeding segment. To reduce the internal forces due to the construction process, a launching nose of 30 m has been used (Fig. 22). The segments at the assembly yard are mounted using temporary supports spaced approximately 10 m (Fig. 21). After welding of all the units that constitute a segment, these temporary supports are removed and the segment is supported by eight moveable units (four supports, and two units per support – one per web, Fig. 24). These moveable units have a maximum vertical capacity of 10000 kN and are guided by a track anchored to the ground. The units are fixed to the structure in the upper part but they slide on teflon sliding pads when the horizontal hydraulic jack, with a capacity of approximately 60 T, is applied. Over the piers, the bridge slides directly over a temporary launching bearing which is fixed to the pier. To ensure a right distribution of the reaction on the box girder webs, launching bearings can rotate in all direction with a vertical axis.

Not all the piers are located below the longitudinal girders, as mentioned, and temporary steel piers have been erected to support the steel structure during launching (Fig. 23). The maximum vertical displacement of the nose during launching is 360 mm, and this displacement has governed the elevation of the bridge during construction.

Figure 22. Launching nose of 30 m.

Figure 23. View of the two first segments.

Figure 24. Launching equipment at assembly yard (Ale-Lastra ©).

4.4.3 *General technical data*

The team of engineers that have participated in the design and construction of the bridge are:

Owner	ADIF
Project Directors	ADIF, Alberto Reguero and José L. Torres-Baptista
Project Design	SERCAL, José M. Warleta
Structural Design	PEDELTA, Juan A. Sobrino, Javier Jordán, Juan V. Tirado, Ricardo Ferraz

Figure 25. Temporary piers.

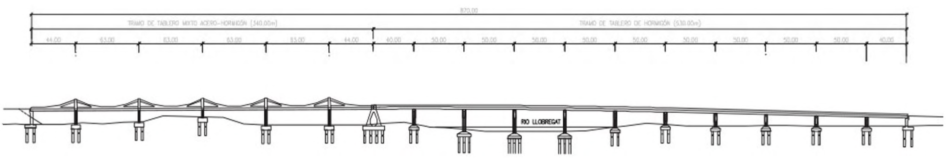


Figure 26. Elevation of Llobregat HSR Bridge.

Site Directors	ADIF, Rafael Rodríguez, Agustín Fernández, Mario García, Juan L. Monjaraz
Site supervision	SERCAL, José L. Aldecoa
Contractor	Constructora Hispánica, Miguel Ruiz
Subcontractors	URSSA, steel erection, Pedro Arredondo ALE-LASTRA, launching, Javier Martínez.

4.5 *Sant Boi Bridge*

4.5.1 *General description*

This high speed railway viaduct is in concept similar to Llinars Bridge but with a different geometry. The overall length of the bridge is 870 m (Fig. 26). The first part of the bridge is a continuous composite structure with 6 spans of 44 + 63 + 63 + 63 + 63 + 44 m (340 m) crossing different infrastructures (Fig. 27); the second part is a continuous prestressed concrete bridge cast in situ – span by span using a travelling formwork – crossing the Llobregat River with a maximum span of 50 m (Fig. 28).

The bridge, 17 m wide, accommodates 2 ballasted tracks, with a platform width of 14 m (Fig. 29). The composite concrete-steel deck consists of parallel transverse I-beams 1 m deep, 3 m apart. The structural system of the deck is similar to the one defined in Llinars bridge.

Longitudinal beams are suspended from curved tied steel box members supported from 11.5 m high steel pylons. The transverse beams are connected with the 1.4 m wide longitudinal box girders having a varying depth of between 3.5 and 5.5 m (Fig. 30). The webs, with thicknesses varying from 15 to 20 mm, are longitudinally stiffened with two or three ½ IPE 500 and transversely every 3.1 m or with diaphragms every 6.2 m.

The cross-section of the pylons is a box-girder with a constant geometry of $2.4 \times 1.5\,m^2$ with plates of 25 mm thickness which are stiffened with standard profiles type ½ IPE 400.

Suspension members have a typical box-girder cross-section with a smooth varying depth of between 1.2 m and 1.8 m and with a constant flange of 1.4 m (Fig. 31). They have been curved

Figures 27 and 28. Llobregat HSR Bridge. Prestressed concrete Viaduct built with a form traveler.

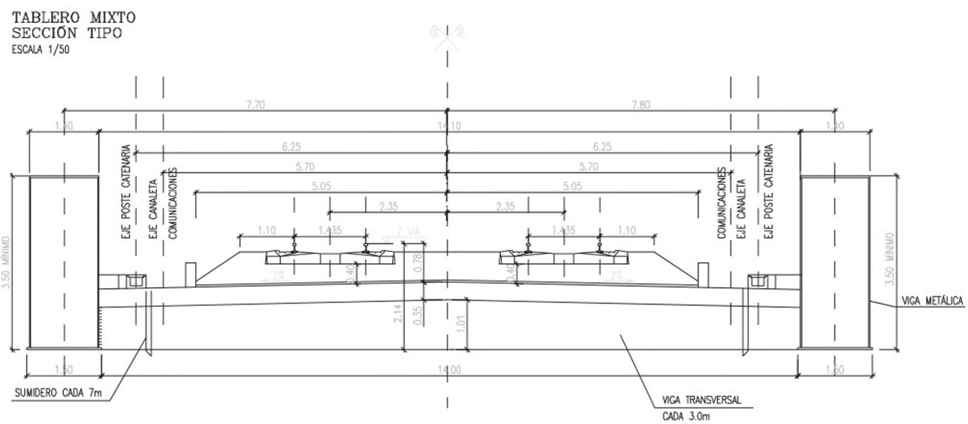


Figure 29. Typical cross-section of the viaduct.

with a radius of 46.7 m to improve the aesthetics of the structure, transmitting a quiet vision of this bridge with a so limited vertical clearance over the motorway. The final steel weight used in the bridge is about 2637 T (554 Kg/m^2 for a platform width of 14 m).

The coating system for the steel protection is similar to the one used in Llinars Bridge. Also the combination of two colours (green and grey) increases the apparent slenderness of the bridge.

A significant horizontal load due to braking and traction of trains and due to track-deck interaction and friction with a magnitude about 25000 kN has conditioned the design of the fixed support (pier 6) between the two structures. As horizontal displacements at deck level should be maximum (30 mm) according to Eurocode, the design of this pier is governed by this serviceability criteria

Figure 30. View of the structural system.

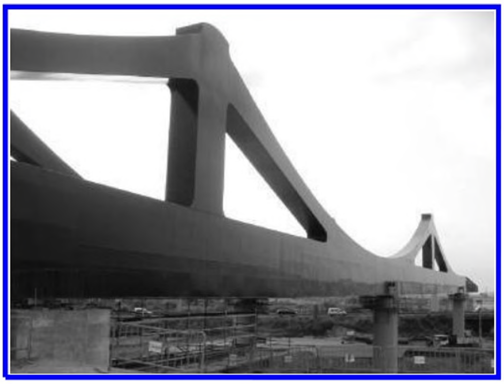


Figure 31. Pylon view.

and the structural configuration adopted is a very stiff element (A shaped reinforced concrete pier founded on 9 piles 2 m in diameter).

The superstructure is supported on concrete piers and abutments and the substructure is founded on piles with a diameter of 2 m. Special attention has been given to the aesthetical design of piers of both parts of the viaduct (Figs. 32 and 33).

4.5.2 *Construction process*

The construction of the steel part of the viaduct was initiated by January 2006 and it is expected to be completed by the end of 2006.

The steel members have been fabricated by different steel yards in Sevilla, Madrid and Barcelona under the coordination of Talleres Torrejón (ACCIONA). As in other steel bridges the units are transported in lengths of approximately 15–20 m, and are assembled at the site on a platform behind the south abutment (Fig. 34).

The bridge is assembled in three segments of approximately 125 + 126 + 89 m. After the assembly of each segment, that takes about twelve weeks, the segment is pushed forward to permit the assembly of the succeeding segment. To reduce the internal forces due to the construction process, a launching nose of 25 m has been used (Fig. 34).

The segments at the assembly yard are mounted using temporary steel supports spaced approximately every 11 m (Fig. 35). After welding of all the units that constitute a segment, 50% of these

Figure 32. Typical pier of the steel viaduct.

Figure 33. Typical Pier of the concrete viaduct.

Figure 34. Assembly of the bridge at the site (first segment before launching).

Figure 35. Tranverse guides for launching.

Figure 36. Launching hydraulic jack at abutment.

temporary piers are removed and the segment is supported by the rest of the temporary supports (6 supports for each longitudinal beam), which allocate the teflon sliding pads during launching. Uniform distribution of vertical load in the two webs is controlled with two vertical hydraulic jacks in each support. These units also include a side guidance system with a roll coinciding with a longitudinal stiffener. The steel structure is pulled from the abutment using two horizontal hydraulic jacks and dywidag bars connected to the bottom flange of the longitudinal beams (Figs. 36–39). Over the piers, the bridge slides directly over a temporary sliding unit supported on POT bearings which permits rotation. The maximum movement of the nose during launching is 270 mm.

4.5.3 *General technical data*

The team of engineers that have participated in the design and construction of the bridge are:

Owner	ADIF
Project Directors	ADIF, Alberto Reguero and Paloma Paco
Project Design	GPO-PEDELTA, Xavier Montobbio, Rafael Domínguez
Structural Design	PEDELTA, Juan A. Sobrino, Javier Jordán, Juan V. Tirado, Ricardo Ferraz, Lara Pellegrini
Site Directors	ADIF, Rafael Rodríguez, Mauro Bravo
Site supervision	GPO-PEDELTA, Antonio Puertas, Antoni Pons
Contractor	ACCIONA, Jaime Vega, Roberto Carballo
Subcontractors	Talleres Torrejón (ACCIONA) Gonzalo Rodríguez, Manuel Sánchez, Marta Calvo.

Figure 37. Assembly of the bridge at the site before the second launching phase.

Figure 38. Assembly of the bridge at the site after the second launching phase.

Figure 39. Aerial view of the bridge almost completed.

5 CONCLUSIONS

Construction of railway bridges for high speed lines represents a significant cost of the network. Due to the important magnitude of vertical and horizontal loads, the design of these bridges requires a judicious selection of the structural configurations and erection procedure. In some cases, the optimization of the use of the construction materials could lead to significant economical savings.

Steel and composite steel-concrete solutions may be competitive compared with classical prestressed concrete decks. The steel presents outstanding mechanical properties, the feasibility to be used in innovative forms and significant advantages during the erection process using light prefabricated elements. The selection of a composite deck suspended from curved steel members has permitted the design of elegant bridges with a strong personality and the use of an incrementally launched erection method has avoided interferences with existing infrastructures below the bridge.

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CHAPTER 7

Prestressed concrete railway bridges

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ABSTRACT: Many long viaducts and long span bridges made of prestressed concrete have been recently designed and built in the new High-Speed railway line Madrid-Barcelona-French Border. The main specific features both the design and construction point of view will be dealt in the article.

1 INTRODUCTION

In recent years, on the High-Speed Railway Line Madrid-Barcelona-French Border, we have designed various viaducts that share common typological characteristics. These viaducts span valleys or rivers and therefore have great lengths and heights with spans ranging from 30 to 66 m. Many of these projects have already been built and others are currently under construction.

The lengths range from 207 m to 1122 m, and their decks are all continuous with box girder cross-sections made of prestressed concrete with maximum pier heights ranging from 12 to 65 m (Figs. 1, 2).

We will focus on the bridge over the Ebro river. Its total length amount to 546 m with the following span distribution: $18 + 6 \times 24 + 60 + 120 + 2 \times 60 + 42$ m. It has an innovative deck in which we tried to adapt the concept of the metallic lattice of large railway bridges to the structural and construction domain of prestressed concrete bridges. The cross section is made of a box girder whose webs contain circular voids and connected by ribs in the upper part. From the structural point of view, this is actually a Vierendeel girder. It has a total depth of 9.15 m. The cross section has a trapezoidal shape. In the upper part the girder maximum width is 16.56 m and the bottom part

Figure 1. Viaduct 4. Madrid-Zaragoza. High speed line. Sub stretch VIII (Spain).

Figure 2. Viaduct 2. Madrid-Zaragoza. High speed line. Sub stretch VIII (Spain).

Table 1. Main features of the viaducts.

Stretch	Viaduct	Length	Span distribution	Max. height	Status
Madrid-Zaragoza Sub-stretch VIII	1	207	$36 + 3 \times 45 + 36$	22	Built
	2	510	$45 + 7 \times 60 + 45$	54	Built
	3	252	$36 + 4 \times 45 + 36$	47	Built
	4	330	$45 + 4 \times 60 + 45$	56	Built
	5	450	$45 + 6 \times 60 + 45$	65	Built
	6	207	$36 + 3 \times 45 + 36$	50	Built
Madrid-Zaragoza Sub-stretch XV	The Huerva River (1)	1122	$2 \times 49.5 + 14 \times 66 + 2 \times 49.5$	48	Built
	The Huerva River (2)	1111	$2 \times 44 + 14 \times 66 + 2 \times 49.5$	48	Built
Lleida-Martorell Sub-stretch VI	The Francolí River	664	$32 + 15 \times 40 + 32$	20	Built
Lleida-Martorell Sub-stretch XI-A	Avernó	810	$45 + 11 \times 60 + 45$	38	Under construction
	Anoia	342	$36 + 6 \times 45 + 36$	21	Under construction
Lleida-Martorell Sub-stretch XI-B	Ca'n Torres	252	$36 + 4 \times 45 + 36$	37	Under construction
El Papiol-San Vicent dels Horts	El Papiol (1) and (2)	570	$45 + 8 \times 60 + 45$	12	Under construction
San Vicent dels Horts-Sta. Coloma	Sta. Coloma (1) and (2)	630	$45 + 9 \times 60 + 45$	13	Designed
Río Llobregat – Costa Blanca	Llobregat River	202	$36 + 48.75 + 32.5 + 48.75 + 36$	15	Designed

it amounts to 12.90 m. The web voids circular of a 3.80 m diameter placed at a distance of 6.00 m from one another.

The construction procedure proposed and used in most of the cases has been that of incremental launching of the segments from one of the abutments. This procedure was deemed the most adequate most cases both due to financial and environmental factors. It is also especially suitable for the range of spans and lengths tackled, particularly in the case of railway bridges, given the capacity of the deck to resist the unfavourable stresses during construction.

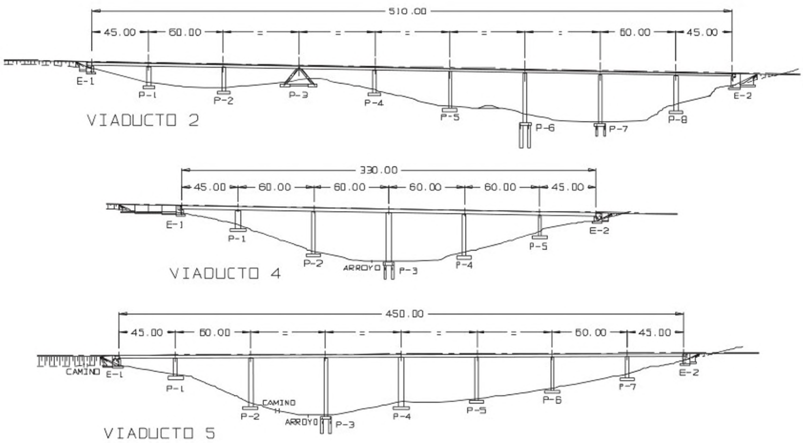

Figure 3. Viaduct 2.4.5. Madrid-Zaragoza. High speed line. Sub stretch VIII (Spain).

In this article we would like to explain the most significant aspects that have conditioned the design of this typology of bridges both geometrically and from the point of view of loads. We also intend to describe the most important characteristics designed to respond to these conditions, as well as to point out the most interesting ones and those specifically related to the resistant response.

2 BASES OF THE DESIGN AND THEIR INFLUENCE IN THE RESISTANT RESPONSE

2.1 *The general criteria of the design*

The high-speed railway imposes very demanding layout parameters, which together with the land characteristics, give rise to a large number of structures. Like in our case, these structures have great lengths and cross valleys at great heights. In order to optimise the span and the number of supports, the spans designed range from 40 m to 66 m.

The design of the viaducts is characteristic for the maximum optimisation of the materials used in the piers and the deck by searching for efficient structural solutions. An economical construction is also taken into account by using the most industrialised procedures possible.

These spans allow us to use a box cross-section, made of prestressed concrete, as the most suitable to resist the significant bending and twisting moments produced in a double track railway bridge. The depths chosen correspond to a 1/17 slenderness. This depth-span ratio establishes an adequate balance between the optimisation of the structure materials and costs and the final finish of the viaducts (Fig. 3).

As for the web arrangement with regard to the tracks, the cross-section was designed so as to minimise the reinforcement for the transverse bending moment.

In the pier design we tried to emphasize the slender character of the viaducts without causing difficulties in the construction.

In railway bridges the construction by incremental launching of box girders with spans between 45 and 90 m is clearly competitive, especially for great lengths and pier heights due to the fact that this procedure does not force us to oversize the deck. This procedure also allows us to perform the deck construction independently, away from the valley or the river, thus minimising the environmental impact of the construction.

2.2 *Layout*

The incrementally launched decks require the viaduct axis to be a circular helix in order to achieve its inscription during the launch.

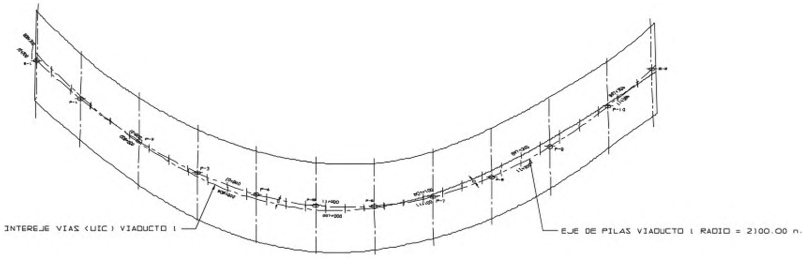

Figure 4. Plan view. Railway axis in a viaduct to be incrementally launched.

On many occasions, the layout conditions do not allow this necessary requirement to be fulfilled, which does not invalidate this construction procedure. Since the tracks are laid over ballast, it is possible to make the geometry of the track independent from the viaduct geometry. There are different possibilities of design depending on the variations in the plan or the elevation of the layout:

a) *Variations in the plan*: The most usual situation is that the axis of the tracks in the viaduct area is totally or partially in a transition curve or alternating straight or circular alignments and transition curves. In these cases, when the alignment in the elevation is straight, there is a possibility of placing the viaduct axis in a circular alignment whose radius and centre can be determined under the condition of minimising the eccentricities between the track and viaduct axes. If these eccentricities are small it will not be necessary to enlarge the platform; but if they are significant, we may need to enlarge the platform considerably. The existence of these eccentricities will condition, as we will explain later, the design of the cross-section and we make us take them into account in the analysis due to the fact that there may be a significant increase of twisting moments both corresponding to the overload and to the permanent load (Fig. 4).
b) *Variations in the elevation*: The possibilities of variation in the plan are more limited since the alignments are straight or vertical parabolic alignments of a great radius. These parabolic alignments can be adjusted to circular curves in order to allow the viaduct to be incrementally launched. If the alignment curve exists only in some of the viaduct ends, its axis may be kept straight thus achieving the variation in the elevation with an increase of the ballast thickness (if it were small). On the other hand, it can also be reached by a protrusion in the structural cross-section of the deck.
c) *Variations in the plan and elevation*: When the axis is circular in the plan and in the elevation it is placed in an alignment, the viaduct cannot be incrementally launched in a horizontal plane, which makes this procedure unfeasible. Nonetheless, it is possible to carry out the incremental launching along a straight alignment in the elevation and after the launching, during the replacement of the provisional launching supports by the definitive ones, we can produce a downwards movement, variable in each pier. In this way, a general bending moment is produced in the whole deck that allows us to couple it with the desired layout. This bending moment is assumed perfectly due to the great value of the radii used. These stresses gradually diminish with the time as a consequence of the creep and shrinkage deformations of the prestressed concrete deck (Fig. 5).

2.3 *Vertical loads*

The designed viaducts have to support the dead load of the structure, dead loads of the track with the ballast, service channels, side-walks and parapets as well as the live loads of the railway. The bases of the analysis establish the following as live loads:

– Load model from the Spanish Standard IPF-75: with the impact coefficient corresponding to a maximum speed of 200 km/h.

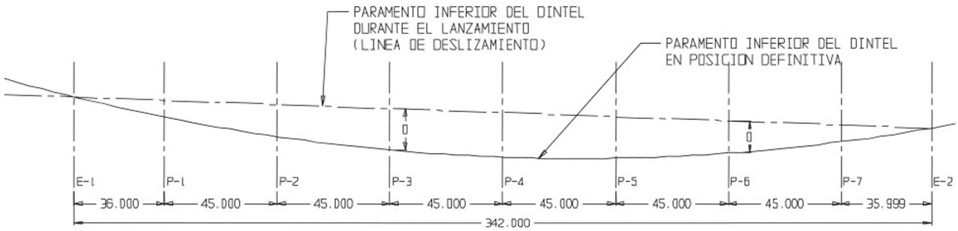

Figure 5. Elevation view. Railway axis in a viaduct to be incrementally launched.

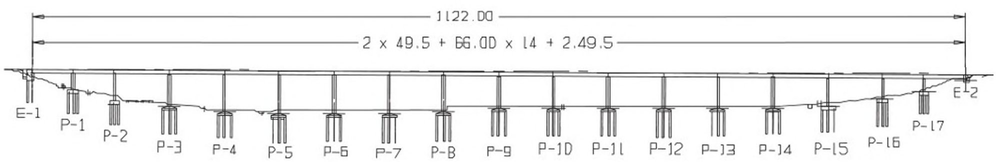

Figure 6. Viaduct over the Huerva River in Zaragoza (Spain).

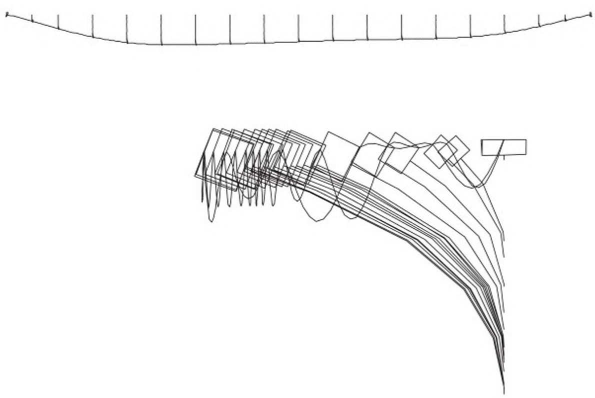

Figure 7. Plan deck and elevation view of the piers under view. Eccentric railway loads.

– Load models from the Eurocode-1 (EC-1): Load model LM-71 and SW-2 with impact coefficients corresponding to a maximum speed of 220 km/h.
– Real Train Loads according to EC-1: with the impact coefficient corresponding to a maximum speed of 350 km/h.

The maximum distribution of the stresses produced by these loads determine the sizing of the elements making the bridge. The existence of two tracks introduces very important twisting moments. To these twisting moments due to the track eccentricity we should add those due to the inscription in the adequate layout as well as those that could be produced by dead load if the platform were not symmetrical to the viaduct axis.

In high viaducts the twisting moments in the deck produce transverse bending moments in the piers, which as a consequence of their flexibility, introduce two effects in the pier-deck interaction (Fig. 6):

– The pier, due to the presence of a guided is restrained by the deck in its transverse displacement and twist, because of complementary horizontal forces (Fig. 7)
– The deck in its turn accumulates twisting moments in the stiffer piers (the shorter ones) and in the abutments, since the twists are not completely restrained in the high piers. On the other

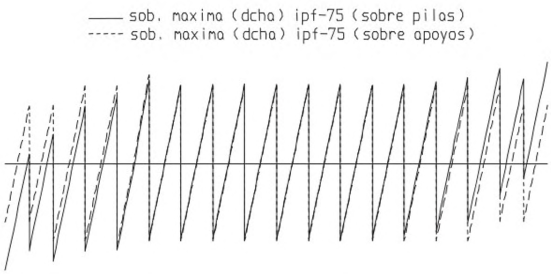

Figure 8. Transverse shear forces under eccentric railway load.

Figure 9. Transverse bending moments under eccentric railway load.

Table 2. Impact coefficients φ' for the displacement in the midspan.

Speed (km/h)	$1+\varphi'$ Dynamic analysis	$1+\varphi'$ Approximate formula
350	1.09	1.19
300	1.05	1.16
150	1.01	1.07
100	0.99	1.05

hand, the accumulated effect of the horizontal loads introduced by the piers, produces not quite negligible bending moments in the vertical axis.

Figures 8, 9 compare the twisting distributions and the bending moments in the vertical axis in the Viaduct 1 over the Huerva River, taking into account or not the pier flexibility.

The dynamic analysis we have done for these bridges shows a very good behaviour. As an example of the response of a large span bridge to the crossing of high-speed train we present here some results of the study we carried out on the Huerva River Bridge. In this case, the dynamic study was carried out with the typical high-speed train defined by the Eurocode, with a total length of 385 m and average weight of 24.4 kN/m. We calculated stresses, displacements and accelerations for four different speeds: 100, 150, 300 and 350 km/h. These speeds were chosen in search of possible resonance effects depending on the distance between the loads and the vibration periods of the structure.

The results regarding displacements are shown in Table 2 in which we represent the values of the impact coefficient for the different speeds and we compare these values with those obtained by applying the approximate formula given by the Eurocode. The coefficient shown is $1+\varphi'$. The effect of the adding φ'' due to the irregularities of the track is not included since in these spans their influence is absolutely irrelevant.

The results of the Table 2 show on the one hand that the formula proposed by the Eurocode for the evaluation of φ' is conservative; The results obtained by the dynamic analysis are more favourable and they are also closer to the reality. On the other hand, the value of the impact coefficient obtained shows that the static calculus carried out on the conventional load model is in fact the one governing the bridge design: a coefficient that increases only by 9% the displacements and stresses obtained for a load four times smaller than the conventional one implies no change in the design established on the bases of static loads.

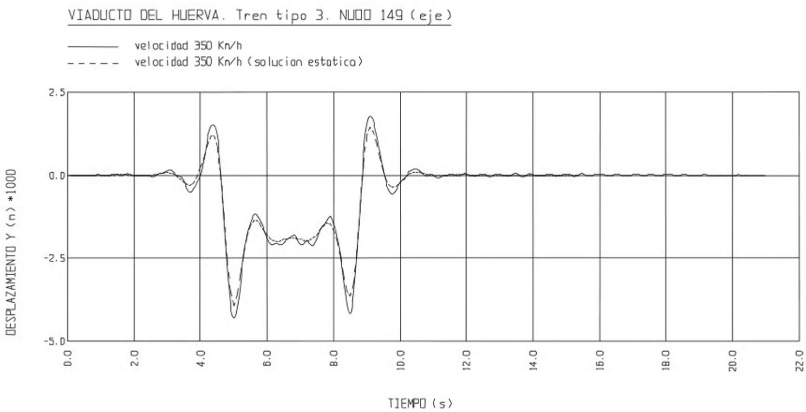

Figure 10. Vertical displacement under static and dynamic loads.

Table 3. Maximum vertical accelerations in the centre of the eighth span.

Speed (km/h)	Maximum vertical acceleration (m/s^2)
350	0.098
300	0.066
150	0.018
100	0.007

These results have been plotted as a time variation of vertical displacements in the centre of the eighth span (Fig. 10).

This displacement is represented in two ways: the result of the dynamic analysis and the result of a cuasi-static analysis (therefore, the effects of inertia and damping are negligible). The differences between the two diagrams are small and in any case they are covered by the impact coefficient value of 1.09 shown in Table 2.

The results of the accelerations are interesting because they show values much lower than the acceptable level (3.5 m/s^2). This is due to the fact that the spans are long and consequently the vibration periods are long as well (Table 3). The time variation reflects most the maximum positions of stresses and displacements (Fig. 11). The maximum values that can be seen both at the beginning and the end of the diagram (for displacements and accelerations) correspond the passage of the engines situated at the front and the rear of the train.

The conclusion of this study is that the dynamic effects in this typology of bridges have little importance and that in any case the Eurocode formula gives a good enough approximation to the solution of the problem.

However in other type of bridges, greater impact coefficient were obtained. A specific dynamic analysis was performed for a continuous prestressed concrete bridge we have recently designed over the Llobregat river in Martorell. It is 202 m long with a main span of 48.75 m and a central V shaped pier. (Fig. 12). Seven different trains configurations were considered: Types 3 and 4 (Eurostar) from Eurocode-1, ICE-2, ETRY, AVE, Talgo AV and Thalys. A Fast Fourier Transform was done in order to find the critical speed at which resonance phenomena could appear taken the most significant vertical modes of vibration.

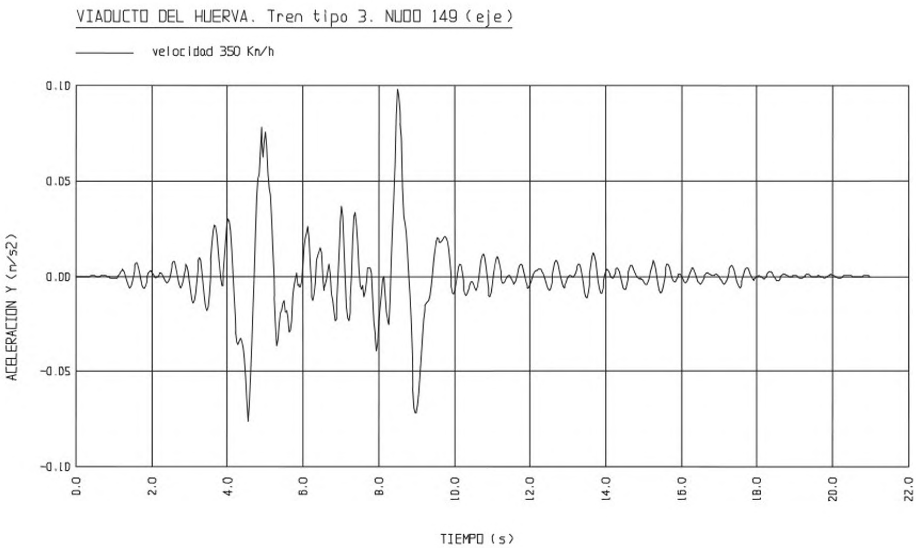

Figure 11. Vertical acceleration.

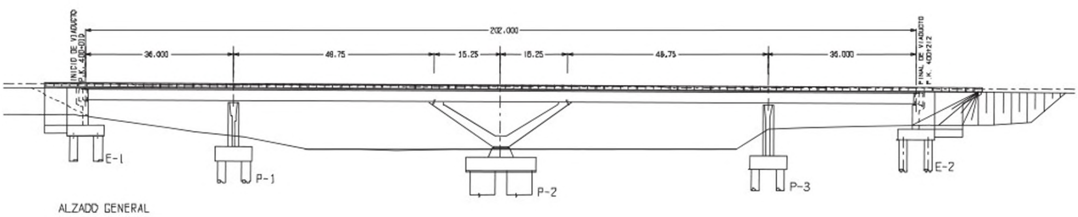

Figure 12. Bridge over the Llobregat river in Martorell (Spain).

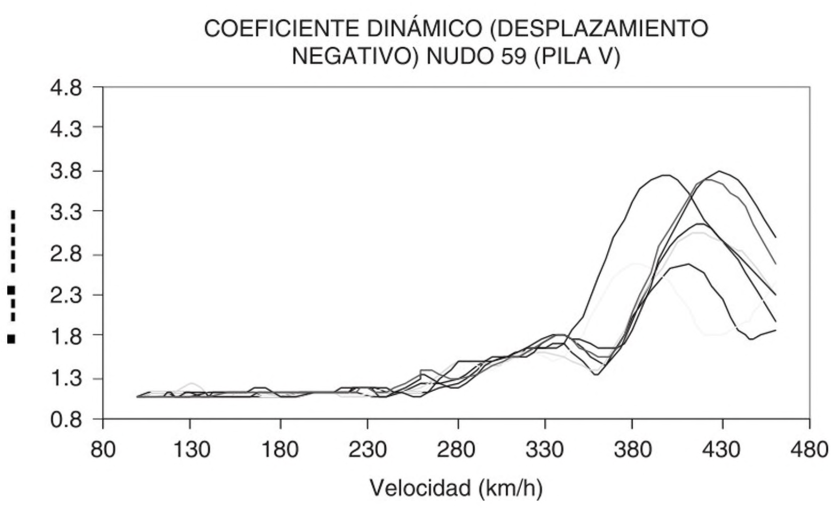

Figure 13. Dynamic coefficient for displacement for different trains and velocities.

A complete dynamic analysis for the seven trains was done with a speed range from 100 to 460 Km/h with 20 Km/h of spacing close to the foreseen critical speeds. A dynamic coefficient of 3.931 in terms of vertical displacements was obtained for the AVE train at V pier section (Figs. 13 and 14). A maximum value of the dynamic coefficient in terms of bending moments of 2.242

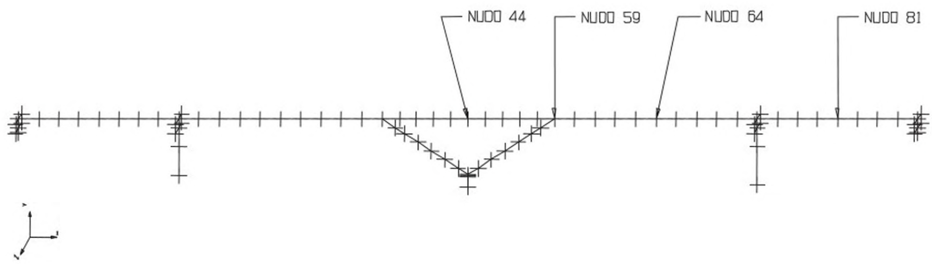

Figure 14. Bar element model.

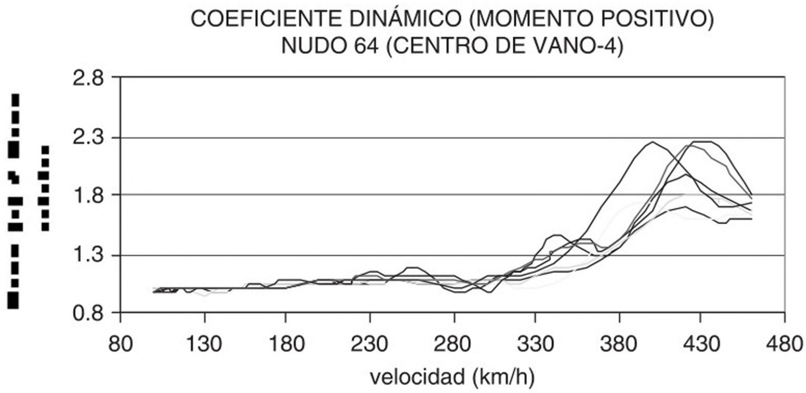

Figure 15. Dynamic coefficient for bending moments for different trains and velocities.

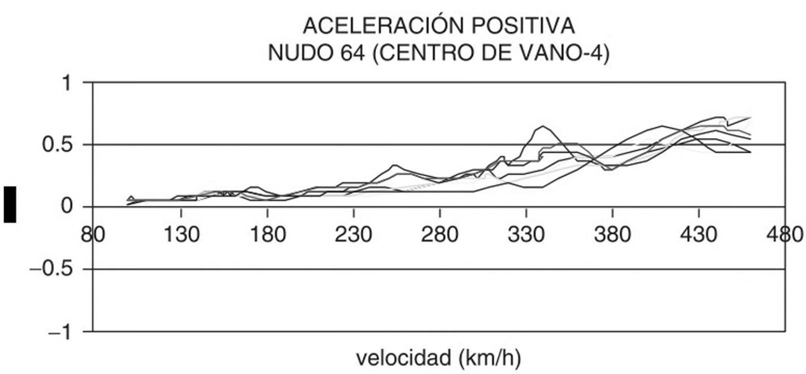

Figure 16. Maximum vertical accelerations for different trains and velocities.

was obtained at midspan of the 4th span for the Talgo AV train (Fig. 15) and a maximum vertical acceleration of 0.733 m/s^2 for the type 3 train of EC-1 at the same section (Fig. 16).

2.4 *Horizontal loads: Track-deck interaction*

The horizontal loads are originated by the climate such as the wind. The wind acts on the whole structure surface exposed, the piers and the deck as well as on the live load itself. These loads produce bending moments of the deck's vertical axis as well as twisting moments which are added to the combined pier-deck effects produced by the live load.

The live loads produces two horizontal loads derived from the accelerations. On the one hand, transverse radial accelerations in curved layouts, which bring about the centrifugal force. On the other hand they bring about longitudinal accelerations produced by braking and traction forces.

The stresses due to the centrifugal forces produce the same effects as those derived from the transverse wind. These forces can be very important for even though the radii are great so is the speed.

The braking forces considered amount to 2 T/m according to EC-1, with a maximum application length of 300 m. The traction forces, in their turn, equal 3.3 T/m with a maximum loaded length of 30 m. Consequently, for long viaducts (longer than 300 m) the maximum force transmitted to the tracks will amount to 700 T. The force transmitted to the deck, bearings and piers will depend on whether the expansion joints are arranged or not on the rail at one or both ends of the deck, as well as on the total length. Within the whole made of the track (rail, sleepers, ballast), the deck, the bearings and the piers as deformable elements with different mechanical properties there are force transferences produced by external loads or deformations imposed, which bring about the well known phenomena of *track-deck interaction* [3]. When the rail is continuous and the track is supported on an element which is not very deformable such as the ground, its sizing depends on the vertical and horizontal loads transmitted by the railway, and the axial forces as a consequence of the deformations restrained by the temperature so the strength reserve to failure is limited. When the tracks are placed over a structure, the imposed deformations produced by the uniform temperature variations and by the creep and shrinkage phenomena in the deck, produce relative displacements between the track and the deck that as a consequence of friction forces with the ballast, produce horizontal loads over both the track and the deck that may exhaust the resistant capacity of the rail. This is the reason why the arrangement of the continuous rail is limited to concrete structures with a total expansion length not greater than 90 m.

With the intention to optimise the design and viaducts construction conditions and track exploitation in continuous viaducts it will be necessary, whenever it is possible, to arrange an expansion joint in one of the abutments. In this way the phenomena of track-deck interaction disappear. Otherwise, the viaducts would have to be sub-divided into smaller lengths with the resulting problems of structural joints, duplication of the bearings and the placing of intermediate stiff and resistant elements in order to resist the horizontal loads in each structure.

If all these factors are studied and properly balanced: the resistant problem for horizontal forces and the track-deck interaction problems, different longitudinal configurations of the viaducts can be obtained (Fig. 17):

– Long continuous viaducts fixed at one abutment with one big expansion joint on the rail at the opposite one.
– Long continuous viaducts with a stiff intermediate pier and two small expansion joints on the rail at both abutments (Viaduct 2 in Substretch VIII).
– Continuous viaducts with a stiff intermediate pier with no expansion joints in the rails. (Martorell Viaduct).

Most of the continuous viaducts we present here are longitudinally fixed in one abutment and have one expansion joint in the rail in the opposite. Since all these viaducts are incrementally launched it is necessary to locate a segment casting yard on one of the abutments. These yards must have the capacity, among other things, to resist the horizontal forces during the launching as well as the loads produced by the wind and temperatures under rest conditions. These are, therefore, elements able to be adapted to become anchorage elements of the deck under service conditions. This is why most of the viaducts presented here, except the Viaduct 2 of the Sub-stretch VIII (Figs. 2,3) and Martorell Viaduct (Fig. 12), are anchored in the abutment corresponding to the segment casting yard. The abutment-yard whole is the structure in charge of transmitting to the ground the horizontal forces produced by the braking and traction loads, the longitudinal wind and those produced by friction forces of the elastomeric-teflon devices as a consequence of the deformations due to the temperature, creep and shrinkage.

The total horizontal forces, under service conditions, are much greater than those corresponding to the situation during construction for the following reasons:

– During construction, the reactions in the supports produced by the horizontal forces correspond to the total permanent load and not self weight.

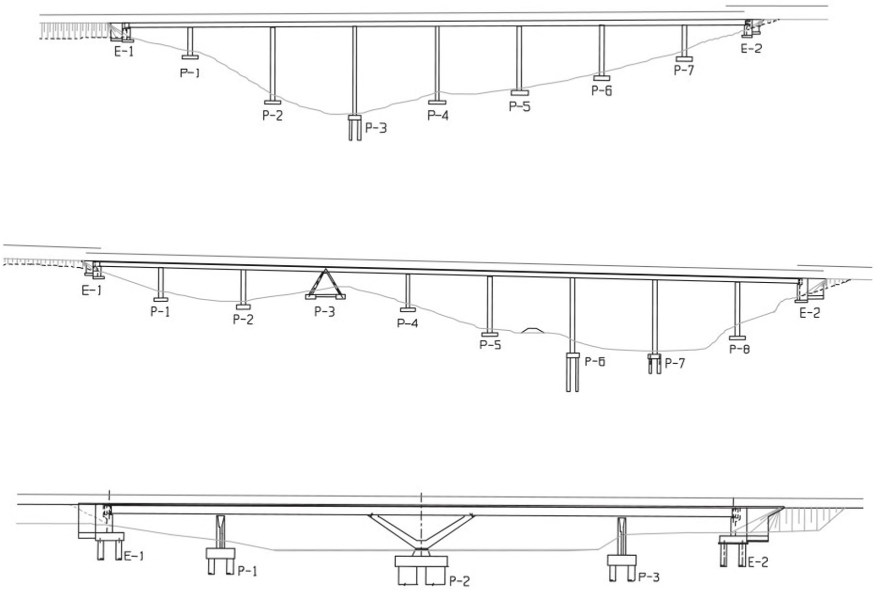

Figure 17. Viaduct resistant schemes for horizontal loads.

- The coefficient of friction of the teflon supports considered under service conditions amounts to 5%, while during construction it amounts to 2.5%. Actually, the value of 2% is hardly reached if special greases are used to reduce this coefficient.
- Under service conditions we must take into account the braking and traction forces for viaducts longer than 300 m reach a maximum value of 700 T.

In spite of the effectiveness of this structural disposition, we must bear in mind the fact that the viaduct length imposes certain limits. On the one hand, the availability of expansion devices able to admit great movements. At present, there are devices with the movement capacity of up to 1200 mm, which establishes a length limit between 1200 and 1300 m for prestressed concrete viaducts. On the other hand there is a limit, though higher than the previous one, which depends on the capacity of the deck to admit the prestressing force that would counteract these horizontal forces.

The concept of the Viaduct 2 of the sub-stretch VIII, mentioned before responds to the need to reduce the movements in the expansion joint and to place the fixed point by a delta shaped pier on a small hill in the valley. This disposition allows us to double the length of the viaducts by placing two small expansion joints in the rail.

In Martorell Viaduct and intermediate V shaped pier was designed. In order to resist the horizontal forces and to reduce the longitudinal movements in this section, the pier is founded over slurry-walls with a big longitudinal stiffness. (Fig. 18). The deck has a total length of 202 m, the expansion length is 101 m (greater than 90 m). In order to avoid the expansion joints at the abutments a complete track-deck interaction analysis was performed. The deck and rails were modelled connected with non linear springs corresponding to the mechanical behaviour of the ballast (Fig. 19). The main conclusions of the analysis were:

- The stress increment in the rail due temperature actions was 42 MPa. (Fig. 20)
- The maximum stress increment in the rail, in compression, due to temperature and braking-traction forces was 77.6 MPa.
- The longitudinal displacement in the deck due to braking-traction forces was 4.27 mm and 5.2 mm taking into account the foundation influence.
- The braking force transferred to the deck is 80 % of the total force applied at the rail level.

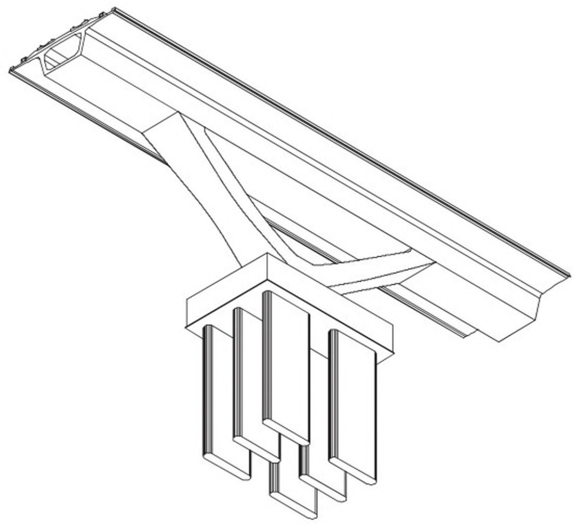

Figure 18. Bridge over the LLobregar River. Deck pier foundation detail.

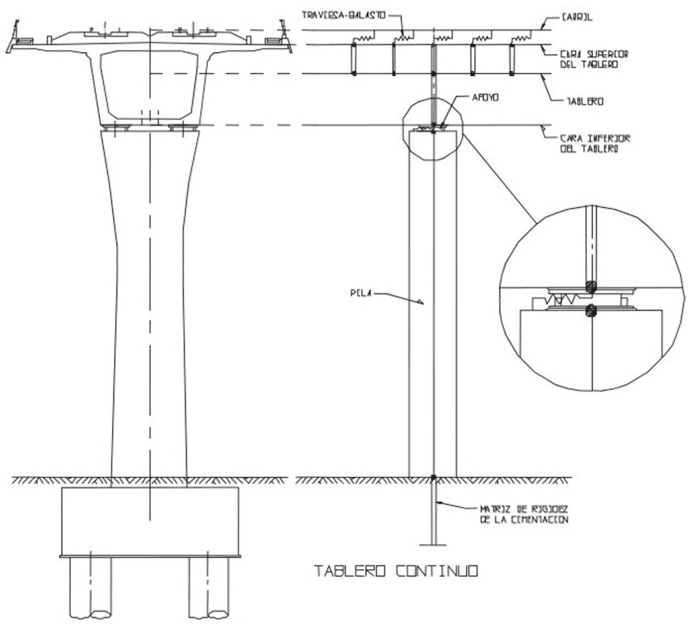

Figure 19. Structural model detail for track deck interaction analysis.

2.5 *Movement limitations*

The Eurocode EC-1 establishes strict limitations for the movements on both static and dynamic levels that must occur in the structure during the passage of the live load at the maximum speed. These limitations are meant to guarantee minimum comfort conditions of the passenger, avoid derailments due to uplift of the wheels or avoid the increment of stresses in the rail as a consequence of the relative movement between these two elements. The limitations affect the maximum vertical acceleration of the deck, its twisting, the rotations in the ends and the horizontal and vertical displacements. These conditions are completely verified for the deck typology and the spans proposed.

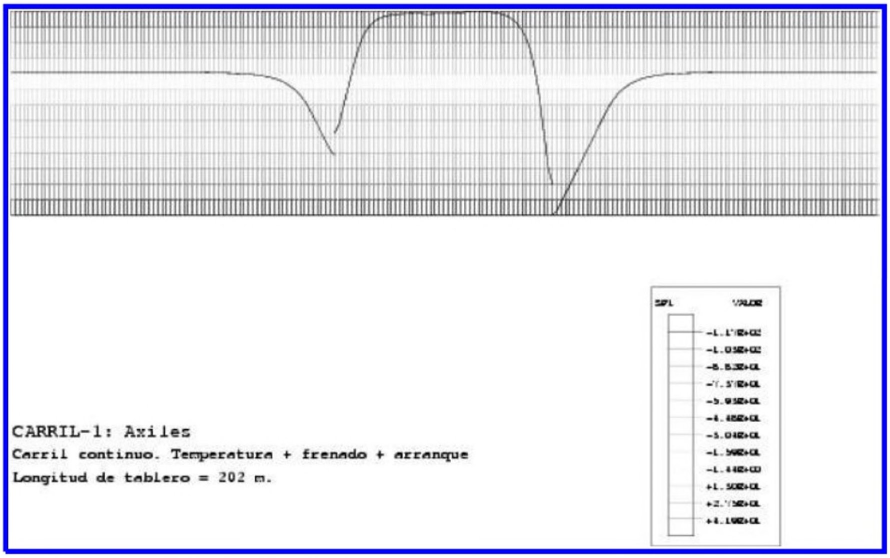

Figure 20. Stresses in the rail due to temperature loads.

3 GENERAL MORPHOLOGY

3.1 *The deck*

The design of the cross section responds to the criteria of performance efficiency before transverse bending moments as well as in the incremental launching procedures. The cross section is a box girder of a constant depth whose value depends on the span, with the slenderness values of 1/17, 2.6 m for 45 m long spans, 3.5 m for 60 m and 4.0 m for 66 m long spans (Fig. 21).

The bottom slab width is 5 m, while the usual disposition of the upper slab is 6.5 m. These dimensions achieve, with inclined webs, to arrange the axes of each track as close as possible to the web axes, minimising the transverse bending moments of the upper slab, and at the same time reducing the dimensions of the bottom slab.

The cantilevers must thus resist the dead loads of the ballast as well as elements like channels for cables, foot path loads and the accidental actions that come from the displacement of the live load in a hypothetical derailment.

In the cases when the track axis does not coincide with the viaduct axis, due to the aforementioned layout, the dimensions of the upper slab must be increased in order to avoid the loads directly transmitted by the track to be applied in the cantilever thus producing a significant increase of the local stresses and problems of displacements and vibrations. (Fig. 22). The upper slab span length and the displacements of the track upon it produce an increase in the amount of reinforcement for local effects.

The dimensions of the upper slab vary between 0.30 and 0.50 m. The typical thickness values of the bottom slab amount to 0.30 m and of the webs to 0.50 m for 60 m long spans and 0.45 m for 45 m long spans. In order to make the construction easier, especially due to the use of the incremental launching, the dimensions are kept constant in the greater part of the deck length, concentrating the geometrically unique features, imposed by the resistant conditions and the prestressing design in the area of supports (Fig. 23). In this way, on either side of the support axis along 3.15 m the web thickness grows in order to accommodate the prestressing service cables anchors. The bottom slab thicknesses in 60 and 66 m long spans grow linearly along 5 m to reach 0.70 m. The support diaphragm has a minimum thickness 1.50 m, allowing the height and width of 1.50 m and 2.25 m

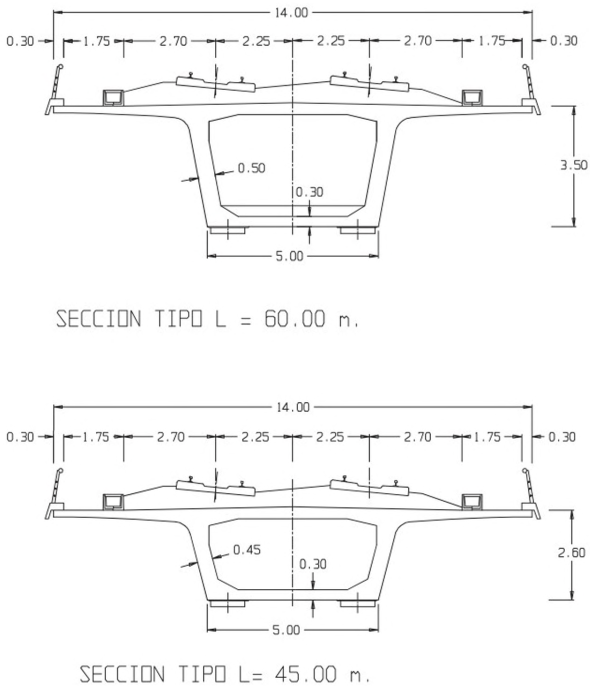

Figure 21. Box girder decks for 60 and 45 m main span.

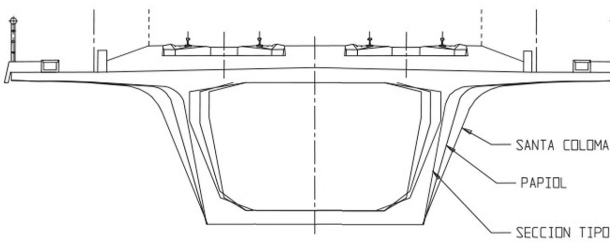

Figure 22. Web variation for different railway eccenticities.

respectively. The diaphragm thickness grows in different piers for long viaducts bearing in mind the long term movements during the bridge's service life.

The distribution of segments was carried out by keeping one segment on the pier and two segments per span, which caused maximum lengths of 15, 20 and 22 m for viaducts 45, 60 and 66 m long respectively.

The prestressing cables are divided into two families (Fig. 24):

- *Cables for Construction*: These are cables of a straight layout arranged in the upper and bottom slab. The cable lengths correspond to three segments and the connection is carried out by using couplers. The tensioning of the cables is carried out in the segment casting yard. The limitation of the slab thicknesses for anchor arrangement makes the cables have a limited number of strands (from 9 to 24 strands of a 16 mm diameter). The distribution of the cable numbers is adapted to the bending moment distribution under construction (Fig. 25).
- *Cables for service conditions*: These are cables with a curved layout arranged all along the web height. The cables are arranged along each span with the anchorages placed in the diaphragms

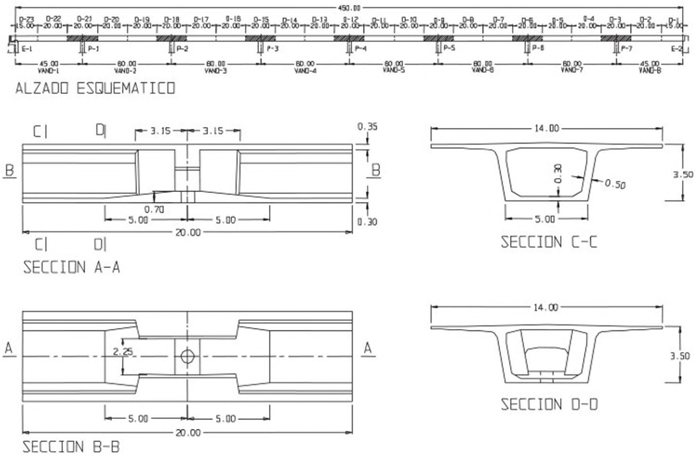

Figure 23. Segment over supports in incrementally launched viaducts.

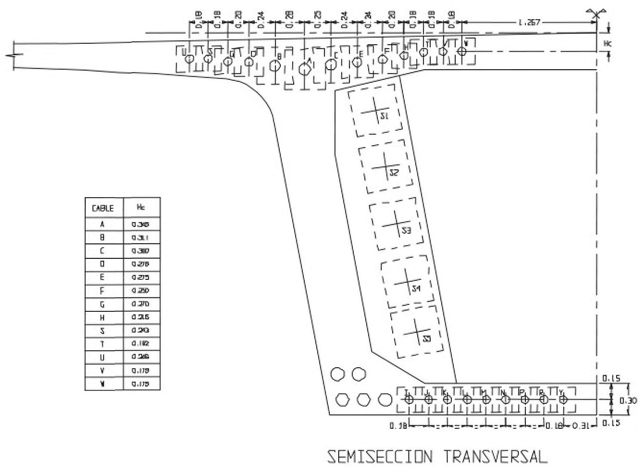

Figure 24. Prestressing cables in incrementally launched viaducts.

in the area of the supports designed for this purpose. The cables are tensioned from both ends once the deck has been launched. In order to decrease the number of sheaths as well as the anchorage devices the number of strands per cable is large (from 31 to 37 strands of a 16 mm diameter, depending on the span). The parabolic layout becomes straight in the anchorage area thus facilitating the crossing of the cables and achieving two things: to minimise the local effects in the area and on the other hand to compensate the loss of eccentricity in the supports area doubling the axil force of the prestressing (Fig. 26).

As for the axil force, the process prestressing is a 56 % of the whole.

The deck anchorage to the fixed abutment is carried out by means of a stopper arranged for this purpose that fits in the rectangular hole within the bottom slab. The coupling is carried out with elastomeric bearings with the necessary rotational capacity (Fig. 27). This system introduces bending moments in the deck cross section that must be taken into account in the analysis. On the other hand, the transmission of the horizontal forces to the webs and the upper slab calls for additional deck length. This arrangement is quite adequate for the construction system of

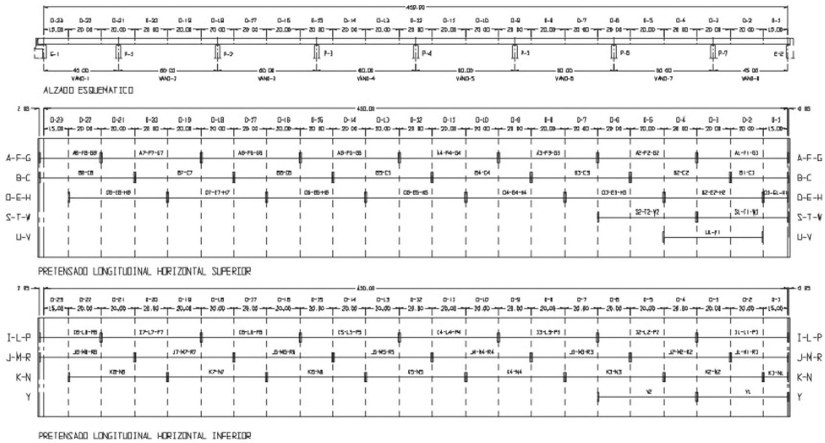

Figure 25. Cables for construction layout in incrementally launched viaduct.

Figure 26. Cables for service conditions anchor blocks in incrementally launched viaduct.

incremental launching because it does not require special anchorage systems while the additional deck length facilitates the launching of the last segment.

3.2 *The piers*

In the pier design we meant to keep the slenderness criteria we applied on the deck. In the piers corresponding to high viaducts we chose the box girder cross section as the most adequate one from the point of view of both resistance and construction. All these piers have a rectangular longitudinal cross section, constant width of 2.40 for viaducts 45 m long and 3.00 for the 60 m long ones. Transversely the dimension varies linearly from 5 m in the upper part, 3.20 m in the waist located 5 m from the upper part and the slope 2.3/100 in the shaft under the waist. The thicknesses of the walls amount to 0.40 m longitudinally and 0.50 m transversely (Fig. 28).

All the piers have great longitudinal slenderness. This is why it was necessary to study their non-linear behaviour. This slenderness and the resulting non-linear behaviour is quite marked in the deck launching stage in which the piers are exposed to the horizontal force corresponding to the friction forces in the sliding bearings without any restraining from the deck unlike the service

Figure 27. Deck to abutment anchor block.

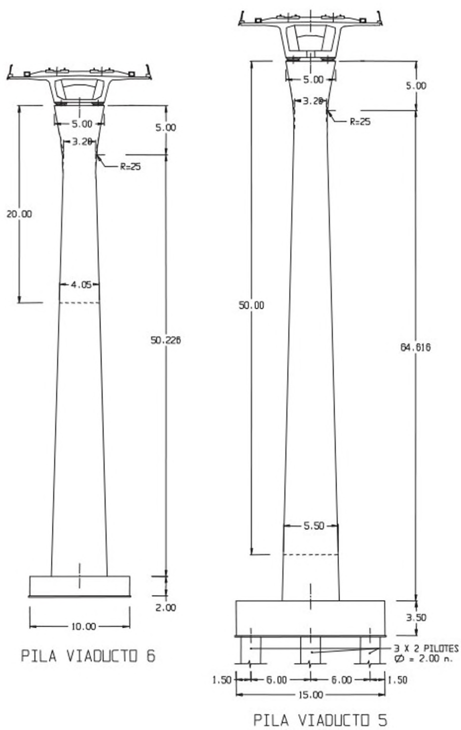

Figure 28. Piers in high viaducts.

condition in which each movement in the pier head requires the surpassing of the friction forces between the supports and the deck.

The non-linear analysis was undertaken from two points of view, the geometric aspect by the updating of the pier deformation and the mechanical aspect by means of the moments-curvature law of each cross section of the pier. The reinforcement schemes of the high piers were conditioned by this non-linear behaviour.

Figure 29. Pier in small height viaducts.

In the pier heads design we took into account the conditions imposed by the use of sliding supports during the launching phase and its later replacement by the final bearings.

The final bearings are made of confined elastomeric teflon in all cases due to the importance of the vertical loads. One of the supports is guided while the other one is free and allows the transmission of horizontal loads of wind and live loads without producing any kind of transversal restraining between them.

For small-height viaducts for river crossings we designed piers with a solid constant circular cross-section quite adequate to cause as little disturbance as possible to the hydraulic regime of the river and at the same to be able to adapt to any skew in the crossing (Fig. 29).

3.3 *The abutments and casting yard*

The abutments geometry is quite variable in order to adapt is as much as possible to the local conditions of the land. In all cases we designed closed abutments to minimise land spilling on the valley which is often very steep. For this same reason it is necessary to arrange long lateral walls (Fig. 30).

As we have already said, the deck is anchored in the abutment on which the segment casting yard is situated. In this abutment it is necessary to co-ordinate the different uses and construction phases.

The yard is made of two large beams joined by a bottom slab thus making a U-shaped cross section, which distributes the vertical loads on the ground. Generally speaking, the yard consists of:

- *Casting area*: where the formwork beams and the deck formwork are located. It is slightly longer than the longest segment (16.50, 21.50 or 23.50 m for the spans 45, 60 or 66 m long).

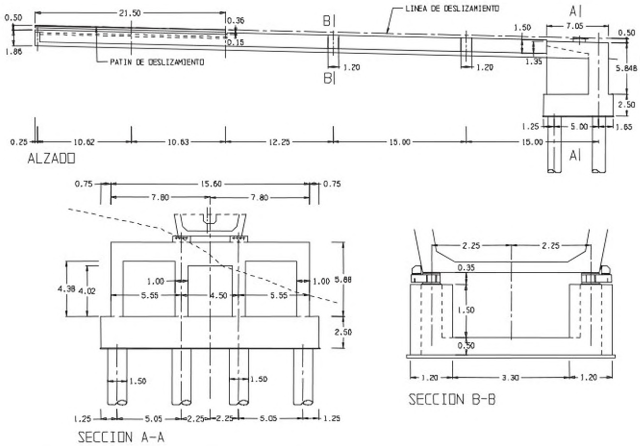

Figure 30. Casting yard in incrementally launched viaducts.

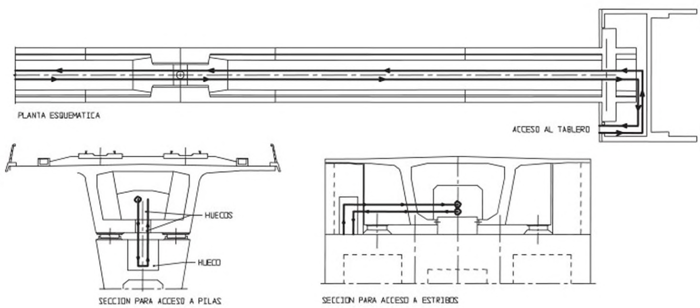

Figure 31. Inspection and maintenance paths layout.

– *Curing Area*: is the area of the yard necessary for the arrangement of two complementary supports per web and its deck is long enough to provide stability in the first stages of the launching.
– *Launching area*: arranged in the front area of the casting yard and the abutment where incremental launching and retaining equipment are located if need may be.

A necessary condition of the yard is the settlement limitations, since their existence could invalidate the deck construction. The structural configuration of the yard, as a great floating beam, makes the calculated settlements smaller than 1 mm even when they are founded over the special granular material of the transition wedges behind the abutment.

If an anchor block is arranged in the launching area, built in the second stage, the structure of the yard can be used as a great *ripper* that transmits the horizontal loads of the deck to the surrounding ground. In order to guarantee the connection with the front wall and control its cracking the girders are prestressed longitudinally.

3.4 *Typefont, typesize and spacing*

In order to guarantee an adequate maintenance of these structures during its service life, the design took into account the access to the main elements. The most prominent details are (Fig. 31):

– Accesses to the interior of the box girder from both abutments by doors arranged for that purpose.

Figure 32. Viaduct over Huerva River under construction.

Figure 33. Long viaducts incremental launching devices developed by ACCIONA.

- Access to the support devices of the abutments and to the corresponding anchor block.
- Pedestrian passages over the deck diaphragms.
- Pedestrian passages across the bottom slab to access the pier heads.
- The pier head area to access their bearings support devices.

4 THE MOST SIGNIFICANT ASPECTS OF THE CONSTRUCTION

All the viaducts described here both those already built and those under construction were carried out by the NECSO construction company (Fig. 32). Although the design foresaw a launching system which used friction jacks, the construction company developed a innovative launching system, [2] based on a maximum automation of the launching process (Fig. 33).

- The sliding elements, the elastomeric teflon sliding pads, remain fixed on the supports which is why it is necessary to arrange a steel plate along the deck sliding axes. In this way there is no need for manual labour for the replacement of sliding pads.

Figure 34. Bridge over the Ebro River. Main span elevation.

- Launching system based on a set of mobile jacks arranged along a tension bar arranged in the casting yard. This launching system has an enormous advantage of being independent from the vertical load existing in the cast yard.
- Automated braking and traction system. This system is connected with to the continuous instruments arranged in the launching jacks and the piers which, in case of any anomaly or dispersion from the expected and programmed values, provokes the interruption and blockage of the launching system.

As we have said, the automatic quality of the launching system is based on the correct instrumentation and control of the critical elements during the launching. The values considered most important are the horizontal forces transmitted to the piers. The unforeseen increase of this value as a consequence of the increase of the friction coefficient or an anomaly in the supports can provoke the breaking of the pier. For this reason, for each pier we calculated the values of rotations angles in the pier head:

- *Emergency Value*: corresponds to the maximum expectable value with no load factor.
- *Break-down Value*: corresponds to the maximum admissible value with the factorised loads reduced by a determined percent.

Both values were determined by the relevant non-linear geometric and mechanical analysis. The emergency value informs the worker in charge of the launching of the proximity of a maximum expectable situation. The break-down value produces the automatic blocking of the system. These values were automatically compared with the rotations measured by clinometres arranged in all the pier heads.

5 A LONG SPAN BRIDGE: BRIDGE OVER THE EBRO RIVER

The bridge over the Ebro River is located on the sub-stretch II-b of the Zaragoza–Lérida stretch on the High Speed Railway Line Madrid-Zaragoza-French Border. (Its total length amounts to 546 m with the following span distribution: $18 + 6 \times 24 + 60 + 120 + 2 \times 60 + 42$ m) [4] (Figs 34, 35).

The unique quality of this typology made us study thoroughly both the methodology and the adequate analysis method. Since this is a deck whose cross section is not homogenous longitudinally, its behaviour is clearly tridimensional which makes it necessary to use the finite elements techniques. This special behaviour is provoked by the presence of circular voids cores in the webs

Figure 35. Bridge over the Ebro River: Internal view.

Figure 36. Bridge over the Ebro River. General view.

as well as the discontinuous upper ribs. In the design phase we analysed a series of innovative aspects both globally and locally, derived from the structure's great due to shear stress deformation.

This specific behaviour was completely confirmed by measurements carried out during construction.

5.1 *Bridge description*

5.1.1 *The deck*

The 546 m total span length is divided into two areas. The first one corresponds to the approach span stretch, it is 162 m long and is made of one 16 m long span and six 24 m long ones. The second area is 384 m long. It becomes a great bridge over the river, made of six spans: 42 m + 60 m + 120 m + 2 × 60 m + 42 m. Both areas are completely united with no joints between them (Fig. 36).

The great Vierendeel girder has a total depth of 9.15 m. The cross section has a trapezoidal shape (Fig. 37). In the upper part it has a maximum width of 16.56 m, while in the bottom part it reaches 12.90 m. The webs have circular voids of a 3.80 m diameter placed at every 6.00 m. The width ranges from 0.50 m and 0.60 m in the area of supports. The bottom slab thickness ranges from

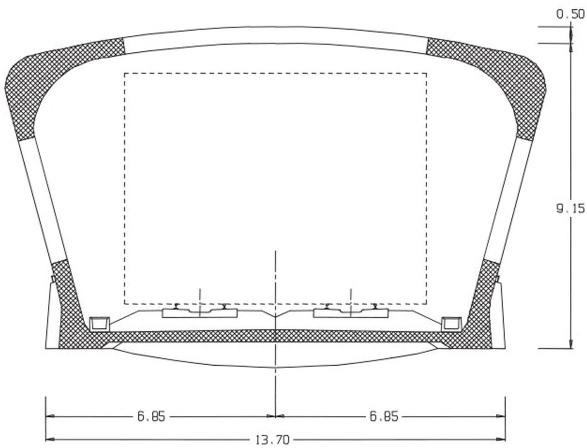

Figure 37. Bridge over the Ebro River. Main spans cross section.

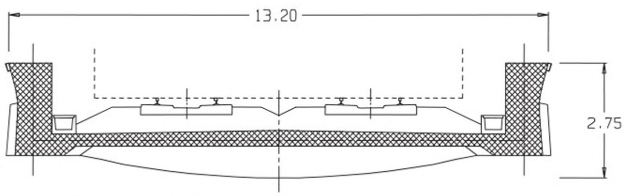

Figure 38. Bridge over the Ebro River. Approach spans cross section.

0.30 m in the point of connection with the webs to 0.39 m in the middle. It has a set of transverse beams with a circular elevation at every 3.0 m with a trapezoidal cross section whose thickness ranges from 0.50 to 0.60 m. In the upper part of the of the box girder we introduce ribs with a circular elevation that follow the direction of the curved upper walls and keep their thickness. They are placed at every 6.00 m and their width amounts to 0.60 m except for those located over the piers whose width is 1.00 m. Over the support axis on the Abutment 2 and in the transition area with the approach spans is 3.30 m wide.

The whole approach span has the same bottom slab as the main span and the same bottom shape in order to automatically establish continuity between these two spans. The lateral girder depth amounts to 2.20 m while the maximum width reaches 1.05 m (Fig. 38).

The construction procedure we incremental launching from both abutments. For this reason the deck was sub-divided longitudinally into segments 18.0, 15.0 or 12.0 m long and one midspan segment 6.0 m long.

The box girder is prestressed both longitudinally and transversely. The longitudinal prestressing is made of three groups of cables:

– *Upper and lower straight prestressing* introduced in the casting yard and tensioned on the front side of the segments.
– *Upper straight prestressing* introduced during construction and tensioned from anchor blocks in the point of connection of the webs with the upper slab.
– *Lower straight prestressing*. It is introduced once the two half-bridges are connected and tensioned from the bottom anchor blocks between the transverse beams.

The transverse prestressing was made of a prestressing inclined in the webs, whose number of cables grows as the cross section approaches the supports. These cables were tensioned once both

Figure 39. Bridge over the Ebro River. Bar element and finite.

half-bridges were incrementally launched due to the fact that the inclined character of the cables is detrimental for the shear resistant capacity in these stages because of its change of sign during the launch. The bottom beams are also prestressed transversely.

The active reinforcement is complemented by the corresponding passive reinforcement.

5.1.2 *The piers*

The bridge has twelve piers. Their shape is trapezoidal and they are made of two separate units of a curved cross section and special elevation (Fig. 39).

The piers corresponding to the approach spans have a maximum width of 2.3 m and they are 10.5 m high. They are composed of two identical units 12 m apart. They are supported on 40 piles whose diameter is 1.5 m.

The piers corresponding to the main bridge are obtained by sections of a single constant cylinder of a curved cross section. Their height is 12 m and the maximum thickness amounts to 4.0 m. The total width in the bottom part is of 22.74 m and they are mutually separated by a variable value of 1 m minimum. They are supported on a variable number of piles of a 2.00 m diameter, four for the pier 7, six for the piers 8, 11 and 12 and twelve for the piers 9 and 10.

Two elastomeric-teflon supports are arranged per pier fixed bearing. Those on the left riverside, according to the launching direction, are guided and those on the right end are unguided. Their size varies depending on the load to which they are exposed.

5.1.3 *The abutments*

The walls are made of one front wall curved in the wings, 50.0 m wide and 10 m high. Two 50 m long longitudinal walls are joined to the front wall that serve as a support for the segment casting yard. The casting yard on the right riverbank is prestressed longitudinally and transversely in order to resist the braking force transmitted by the deck.

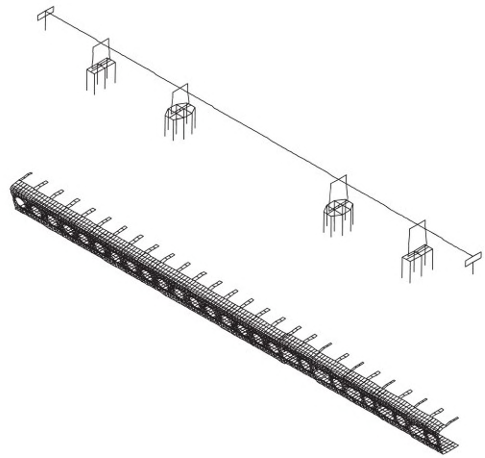

Figure 40. Bridge over the Ebro River. Main piers view. Element model detail.

5.2 *Analysis methodology*

The peculiarity of this typology made us study thoroughly both the methodology and the appropriate analysis elements.

The peculiarities are basically the result of:

- The presence of the circular voids introduces a significant longitudinal deformability in the deck as well as the concentration of more or less unique stresses.
- The discontinuous character of the upper ribs provokes an intermediate structural behaviour between an open U-shaped cross section and a closed box girder cross section.
- In the development of all the studies bar element and finite element models were used (Fig. 40):

The Bar Elements Model

Different models were used for the longitudinal and transverse study of the deck.

The longitudinal study allowed us to obtain the following results:

- The global stresses, both under service conditions and during the construction of the whole made of the deck, bearings, piers, foundations and abutments.
- Longitudinal prestressing study under service conditions and during construction.
- Analysis of the longitudinal reinforcement.

The *transverse study*, carried out with the specific bars models, allowed us to analyse local effects on the ribs and the bottom slab both under service conditions and in a hypothetical accident produced by derailment. This study gave us the sizing of the bottom transverse prestressing and the corresponding passive reinforcement.

The Finite Elements Model

It allows the correct *calibration* of the longitudinal bars model by obtaining the mechanical properties of the cross section.

The study of the stresses concentrations effects due to the presence of circular voids, analysing the adequate shear stress deformation capacity.

Local effects due to the presence of supports, both the final ones over the piers and those used during construction, since all the cross sections of the deck are support cross sections due to the incremental launching.

Study of the stresses in the upper ribs.

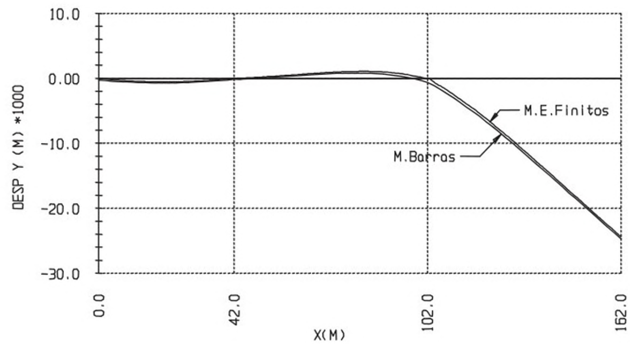

Figure 41. Maximum displacement during construction (bar element and finite element model).

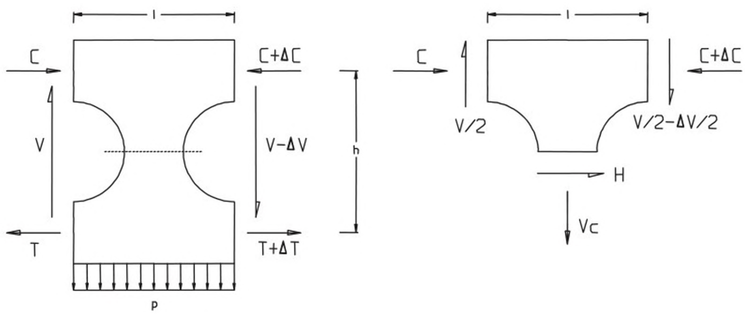

Figure 42. Shear forces resistant scheme.

The structural response in the beam model was very accurate introducing the shear section properties. Very interesting results from this model with shear deformation were obtained related to the increment of longitudinal stresses in the smaller spans and angular discontinuities due to the concentrated loads at the section supports. In Figure 41 is shown both the bar and finite element models, this discontinuity and the good fit from both models.

5.3 *Shear stress deformation*

The presence of circular voids introduces important discontinuities in the transmission of the tangential stresses produced by shear stress and the twisting moment. The resistance mechanism observed is well known and responds to the same behaviour of the Vierendeel girders or hollow web girders. The idea of global equilibrium between the stresses and the external loads between two hollow cross sections clarifies well the structural behaviour of these typologies.

We consider a girder element between two voids at a distance l (Fig. 42) with the stresses acting in both cross sections, the shear stress V, the bending moment decomposed in a pair of compression forces (C) and tension (T), separated h, by the distance between the tension and compression chords. A uniformly distributed load p is considered. The stresses in one cross section differ from those in the other one by an increment Δ.

Such a simple scheme allows us to deduce the set of both active and passive reinforcements for the global shear stress deformation:

– The total transverse reinforcement must resist the horizontal shear force H.
– The previous reinforcement must be incremented to resist tension forces V_c produced by the application of the external loads on the bottom slab: suspension reinforcement.

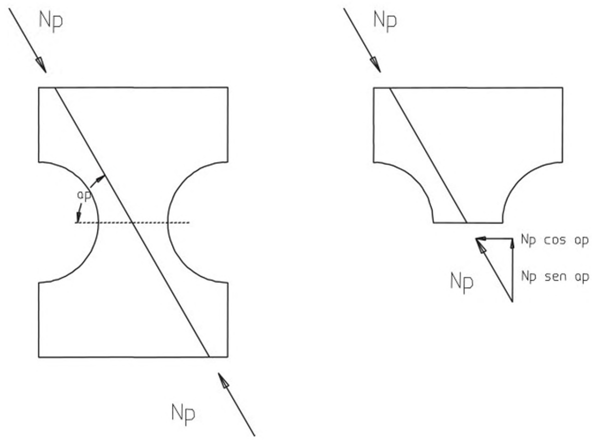

Figure 43. Transverse prestressing structural effect.

– The force H produces bending moments in the end of the void which explains the concentration of stresses within them.

From this perspective, the favourable effect of the transverse inclined prestressing of the web can be analysed (Fig. 43).

The transverse compression N_p produces a reduction of the shear force H by a magnitude $H_p = N_p\cos\alpha_p$. In this case the prestressing still plays a double role: as a load it reduces the shear force and the vertical tension and as a reinforcement it allows us to count on an additional vertical reinforcement.

The presence of alternative shear stresses during the construction by incremental launching made us use a partial prestressing made of an active reinforcement for the final situation and the passive reinforcement during construction, due to the fact that the inclination of the cables would produce unfavourable effects during these stages.

5.4 *Local effects*

Beside the general effects studied, the concentrated loads produce very important effects especially during the stages of incremental launching during construction as well as those derived from the loads concentrated in the piers in their final position.

The incremental launching procedure makes all the cross sections of the deck support cross sections which requires correct sizing of the transverse reinforcement. The presence of circular voids introduces important tensions that were studied using the tridimensional finite elements model.

In order to control the cracking during these stages, one of the straight longitudinal prestressing cables was placed near the above mentioned void.

Another significant tridimensional effect appears in the supports cross sections. The absence of intermediate diaphragms in the supports provokes the appearance of significant transverse bending moments that must be superposed on the vertical ones of the whole deck.

In Figures 44 and 45 we can observe the different behaviour of the supports cross section and an intermediate cross section of the deck. These differences are manifested in the transverse bending of the web and the upper ribs.

The ribs are placed in most of the cross sections completely compressed by the transverse bending moment effects except for those near the supports that become tensioned.

Figure 44. Deck deformed shape under maximum loads.

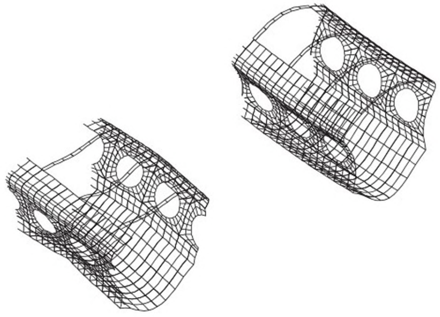

Figure 45. Deck deformed shape at intermediate section and at support section.

This is due to the fact that in the intermediate cross sections the flows of shear stresses produce local effects due to the presence of distributed loads, which cancels the general effects Figure 46. However, the presence of concentrated loads makes the transverse bending effects produced by the shear stress flows add up. These effects are more important than the local bending ones due to the relative importance of the concentrated loads in the supports.

5.5 *Resistant behaviour during construction*

The construction procedure was that of incremental launching of the deck from both abutments (Figs. 47, 48). During the launch, the deck was exposed to stress states that were more unfavourable in certain parts than in the final situation. The measurements of displacements and reactions allow us to verify the hypotheses on its resistant behaviour established during the design.

One of the aspects that was completely verified was the shear stress deformation. During the launch all the sections become support sections with the sign of the shear stresses changing before reaching the pier and after passing it. Since the transverse cross section of the web was designed so as to take the greatest possible advantage of it under service conditions, with an inclined layout, the application of this prestressing would imply quite unfavourable stresses states. This is why the deck was built as a reinforced concrete structure from the point of view of the webs, with the necessary transverse passive reinforcement. These situations produced a significant increment of the displacements, in the component corresponding to the shear stresses as a consequence of cracking which was constantly controlled by the passive reinforcement arrangement. With the displacement data obtained we calibrated the component of the shear area that implied a reduction of up to 10%.

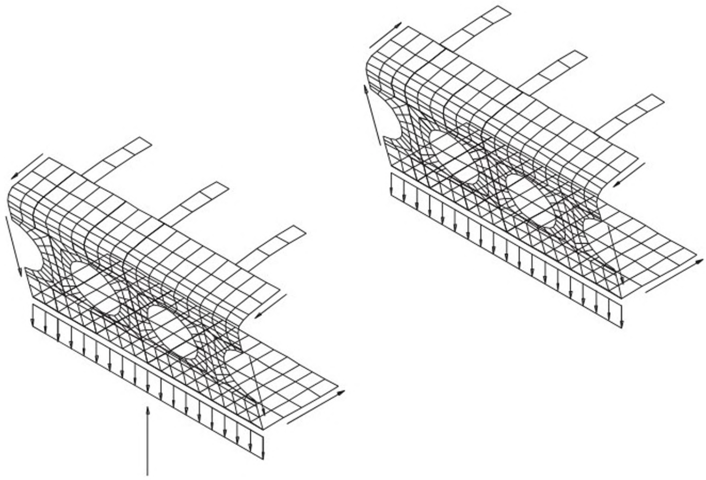

Figure 46. Shear forces scheme at intermediate section and at support section.

Figure 47. Bridge over the Ebro River under construction (1).

Figure 48. Bridge over the Ebro River under construction (2).

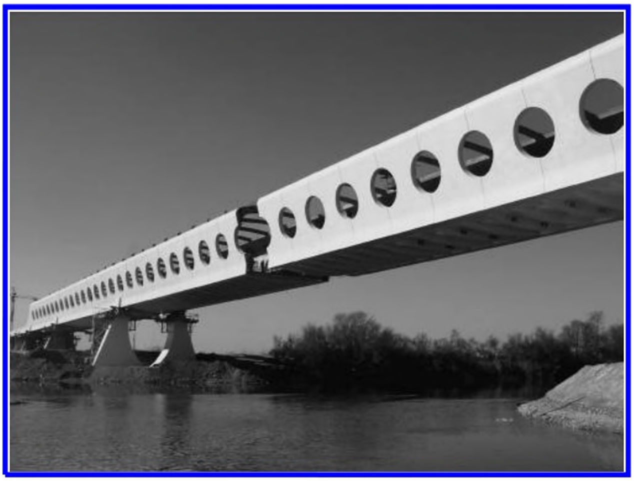

Figure 49. Bridge over the Ebro River before construction of midspan segment.

Once the transverse prestressing cables were tensioned in the main span, the displacements corresponded to an adjustment quite adequate to the non-cracked model. This model with the displacements was interesting for predicting the situation of reaching the midspan and for the adequate sizing of the levelling and blocking mechanisms for the casting of the midspan segment (Fig. 49).

The reactions measured corresponded to the predicted values in all the piers except for slight discrepancies in the pier P-7 that were justified as a consequence of a small settlement in the segment while it was in the casting yard in order to carry out a geometrical adjustment in the transition of the two decks. The displacement values measured in the different load test hypotheses were completely satisfactory.

6 CONCLUSIONS

- We have presented the most important aspects in the long span viaducts designed for high-speed railway lines, particularly emphasising the typological aspects and their influence in the specific resistance problems of this type of structures.
- The most significant resistant aspects of the Ebro River Bridge are those derived from its tridimensional configuration:
  - The longitudinal behaviour can be studied with the model of bars with shear stress deformation. These models take into account the angular discontinuities due to the presence of concentrated loads and modifications in the secondary stresses due to the shear stress deformation.
  - For the study of local effect produced by the presence of concentrated loads as well as the dimensioning of the discontinuous elements and the upper ribs, 3D finite elements model must be used.
  - Most of the specific aspect of its resistant behaviour were verified during construction.

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CHAPTER 8

Dynamic behaviour of bridges due to high speed trains

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ABSTRACT: The railway bridges are exposed to the dynamic effects of high speed trains. As they are composed by a long sequence of axle forces or their groups in regular distances, the bridges are brought into intensive vibration under certain conditions. Therefore, a simple theoretical model of a railway bridge was put together in the form of a Bernoulli-Euler beam subjected to a row of axle forces moving at constant speed. The solution of the governing differential equation results in the deflection-, bending moment- and vertical acceleration-time histories. Moreover, the stresses were classified using the rain flow counting method to receive the stress spectra. The numerical calculations were performed for both the concrete and steel bridges of spans 5 to 50 m subjected to the three types of high speed trains (ICE, Eurostar/TGV and Talgo) running at speeds 5 to 500 km/h. The effect of several parameters was studied in detail in particular the effect of speed, span, damping, material (concrete, steel), critical velocities, vertical acceleration, etc. The stress spectra show the statistical distribution of stress cycles which are important for the fatigue calculations and for the estimation of the bridge life. The simplified formulas for the design of bridges as well as a method for the interoperability calculations are also suggested.

1 INTRODUCTION

The modern technologies in West Europe and East Asia introduced the high speed trains in the passenger transportation. They move at the speed about 300 km/h at these days. In general, the speed over 200 km/h is designed as the high one, however, the new high speed lines are constructed for the speed 350 km/h and the bridges with a 20% reliability for 420 km/h. Therefore, the calculations are often performed up to 500 km/h because this speed has been reached at some tests.

The new transport technology brought, of course, some new technical problems. The dynamics of bridges on high speed lines belongs to them. The maintenance services of French and German Railways have noticed a destabilization of ballast on bridges of small and medium spans on high speed lines. It has appeared a new technical expression for that phenomenon, i.e. "white ballast", which has arisen by milling of ballast grains during the intensive vibration. This phenomenon results in serious consequences: destabilization of ballast, possibility of derailment, deterioration of both the train riding and passenger comfort and at last the increasing maintenance costs. Figure 1 shows an example of this type of bridge vibration [1].

Therefore, the European Rail Research Institute studied the problem, [2], and initiated the present research. Its purpose is to suggest a simple theoretical model, to show the effect of important parameters and the statistical distribution of stresses under high speed trains, to suggest the formulas for a rough and quick design of bridges and to formulate the quantified interoperability condition.

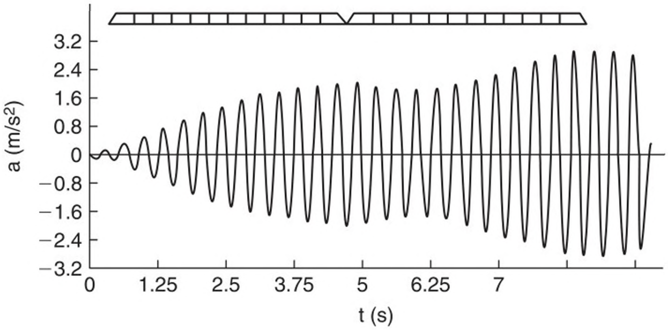

Figure 1. Acceleration-time history of a SNCF bridge of span 38 m subjected to the coupled TGV trains at speed 192 km/h, [1].

2 SIMPLE THEORETICAL MODEL

The Bernoulli-Euler beam represents the simplest theoretical model of a railway bridge, [3]. Therefore, let us assume a beam of span l (Fig. 2) subjected to a row of axle forces $F_n, n = 1, 2, 3, \ldots, N$, which are moving with the constant speed c along the simply supported beam of span l, N – being the number of axle forces in the train.

The following partial differential equation describes the dynamic behaviour of the beam

$$EI\frac{\partial^4 v(x,t)}{\partial x^4} + \mu\frac{\partial^2 v(x,t)}{\partial t^2} + 2\mu\omega_d\frac{\partial v(x,t)}{\partial t} = \sum_{n=1}^{N} \varepsilon_n(t)\delta(x - x_n)F_n, \tag{1}$$

where denotes:
$v(x, t)$ – vertical deflection of the beam at x and time t,
E – modulus of elasticity of the beam,
I – constant moment of inertia of the beam cross section,
μ – constant mass of the beam per unit length,
ω_d – circular frequency of the beam damping,
$\vartheta = \omega_d/f_1$ – logarithmic decrement of damping,
f_1 – the first natural frequency of the unloaded beam,

$$\varepsilon_n(t) = h(t - t_n) - h(t - T_n), \tag{2}$$

$$h(t) = \begin{cases} 0 & \text{for} \quad t < 0 \\ 1 & \text{for} \quad t \geq 0 \end{cases} \tag{3}$$

– Heaviside unit function,
$t_n = d_n/c$ – time when the n-th force F_n enters the beam,
$T_n = (l + d_n)/c$ – time when the n-th force F_n leaves the beam,
$\delta(x)$ – Dirac delta function describing the single concentrated force,

$$x_n = ct - d_n,$$

d_n – distance of the n-th force F_n from the first one F_1, $d_1 = 0$.

The boundary and initial conditions of a simply supported beam are

$$v(0, t) = 0, \quad v(l, t) = 0, \quad v''(0, t) = 0, \quad v''(l, t) = 0, \tag{4}$$

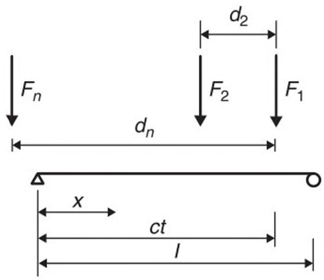

Figure 2. Movement of a row of forces F_n along a simply supported beam of span l.

$$v(x,0) = 0, \quad \dot{v}(x,0) = 0, \tag{5}$$

where the dashes and points represent the derivation with respect to x and t, respectively.

Consequently, the natural frequencies, ω_j, f_j, and forms of natural vibration $v_j(x)$ of a non-damped simple beam are, [3]:

$$w_j^2 = \frac{j^4\pi^4}{l^4}\frac{EI}{\mu}, \quad f_j = \frac{\omega_j}{2\pi}, \quad j = 1, 2, 3, \ldots, \tag{6}$$

$$v_j(x) = \sin\frac{j\pi x}{l}. \tag{7}$$

The method of integral transformation will be applied to the problem, [4]. The mutual Fourier integral transformations of $v\,(x, t)$ are

$$V(j,t) = \int_0^l v(x,t)v_j(x)dx, \tag{8}$$

$$v(x,t) = \sum_{j=1}^{\infty}\frac{\mu}{V_j}V(j,t)v_j(x), \tag{9}$$

where using (7) and (4)

$$V_j = \int_0^l \mu v_j^2(x)dx = \mu l/2. \tag{10}$$

The Laplace-Carson integral transformations are defined, [4]:

$$V^*(j,p) = p\int_0^{\infty} V(j,t)\exp^{-pt}\,\mathrm{d}t, \tag{11}$$

$$V_j(j,t) = \frac{1}{2\pi i}\int_{a-i\infty}^{a+i\infty}\frac{V^*(j,p)}{p}\exp^{tp}\,\mathrm{d}p. \tag{12}$$

The application of the Eq. (8) to the Eq. (1) gives

$$\ddot{V}(j,t) + 2\omega_d\dot{V}(j,t) + \omega_j'^2 V(j,t) = \sum_{n=1}^{N}\frac{F_n}{\mu}\varepsilon_n(t)v_j(ct-d_n), \tag{13}$$

and the transformation (11) of the Eq. (13) yields

$$V^*(j,p) = \sum_{n=1}^{N}\frac{j\omega F_n}{\mu}\exp^{-pd_n/c}F(p)[1-(-1)^j\exp^{-pl/c}], \tag{14}$$

where it was denoted

$$F(p) = \frac{p}{(p^2 + j^2\omega^2)[(p+\omega_d)^2 + \omega_j'^2]} \tag{15}$$

and the damped natural frequency and forced frequency are

$$\omega_j'^2 = \omega_j^2 - \omega_d^2, \tag{16}$$

$$\omega = \frac{\pi c}{l}. \tag{17}$$

Deriving the Eq. (14) the initial conditions (5) and the transformation of the Heaviside unit function appearing in (2) were applied with respect to Eqs (27.6) and (27.10) from [4].

The inverse Laplace-Carson (12) and Fourier (9) transformations present the solution in the following form

$$v(x,t) = \sum_{j=1}^{\infty}\sum_{n=1}^{N} v_0 \frac{F_n}{F} j\omega\omega_1^2 [f(t-t_n)h(t-t_n) - (-1)^j f(t-T_n)h(t-T_n)] \sin\frac{j\pi x}{l}, \tag{18}$$

where the Laplace-Carson inverse transformation of Eq. (15) using (27.41) from [4] is

$$f(t) = \frac{1}{\omega_j' D}\left[\frac{\omega_j'}{j\omega}\sin(j\omega t + \lambda_1) + \exp^{-w_d t}\sin(\omega_j' t + \lambda_2)\right] \tag{19}$$

and the following notations are used

$$v_0 = \frac{2F}{\mu l \omega_1^2} = \frac{2Fl^3}{\pi^4 EI} \approx \frac{Fl^3}{48EI} \tag{20}$$

for the deflection of the beam center due to the force $F = F_n$ applied at the same point,

$$D^2 = (\omega_j^2 - j^2\omega^2) + 4j^2\omega^2\omega_d^2, \tag{21}$$

$$\lambda_1 = \arctan\frac{-2j\omega\omega_d}{\omega_j^2 - j^2\omega^2}, \quad \lambda_2 = \arctan\frac{-2\omega_d\omega_j'}{\omega_d^2 - \omega_j'^2 + j^2\omega^2}. \tag{22}$$

The first term on the right hand side of Eq. (19) expresses the forced vibration due to the moving forces while the second term denotes the free damped vibration, respectively. The step by step entrance of individual forces F_n to the beam at time t_n as well as their departure at time T_n are described by the Heaviside unit functions $h(t)$ in Eq. (18) as well as their shifts in time $t - t_n$ and $t - T_n$, respectively. Thus, the Eq. (18) represents the solution of the problem in a closed compact form.

The bending moment of the beam (proportional to the stress) may be calculated in a similar way

$$\begin{aligned} M(x,t) &= -EI\frac{\partial^2 v(x,t)}{\partial x^2} \\ &= \sum_{j=1}^{\infty}\sum_{n=1}^{N} M_0 \frac{F_n}{F} j^3\omega\omega_1^2 [f(t-t_n)h(t-t_n) - (-1)^j f(t-T_n)h(t-T_n)] \sin\frac{j\pi x}{l}, \end{aligned} \tag{23}$$

where

$$M_0 = \frac{2Fl}{\pi^2} \approx \frac{Fl}{4} \tag{24}$$

Table 1. Bridge parameters.

Material	Span l(m)	5	10	15	20	30	50
Concrete	f_1 (Hz)	32.35	15.09	9.66	7.04	4.51	2.57
	ϑ(l)	0.63	0.34	0.23	0.18	0.18	0.18
	G (kN)	250	600	1050	1600	3000	6000
Steel	f_1 (Hz)	19.12	11.77	8.86	7.25	5.46	3.82
	ϑ(l)	0.64	0.23	0.12	0.08	0.08	0.08
	G (kN)	125	300	525	800	1500	3000

is the bending moment at the centre of a simple beam due to the force $F = F_n$ applied at the same point.

The vertical acceleration of the beam is deciding for the ballast stability and yields

$$\begin{aligned} a(x,t) &= \frac{\partial^2 v(x,t)}{\partial t^2} \\ &= \sum_{j=1}^{\infty}\sum_{n=1}^{N} v_0 \frac{F_n}{F} j\omega\omega_1^2 [\ddot{f}(t-t_n)h(t-t_n) - (-1)^j \ddot{f}(t-T_n)h(t-T_n)] \sin\frac{j\pi x}{l}, \end{aligned} \tag{25}$$

where

$$\ddot{f}(t) = -\frac{\omega_j'^2 - \omega_d^2}{\omega_j' D}\left[\frac{j\omega\omega_j'}{\omega_j'^2 - \omega_d^2}\sin(j\omega t + \lambda_1) + \exp^{-\omega_d t}\sin(\omega_j' t + \lambda_3)\right], \tag{26}$$

$$\lambda_3 = \lambda_1 + \arctan\frac{2\omega_d\omega_j'}{\omega_j'^2 - \omega_d^2}. \tag{27}$$

When the derivatives in Eq. (25) are calculated, the zero time derivations of the Heaviside function (3) were taken into account.

3 NUMERICAL CALCULATIONS

The Eqs (18), (23) and (25) were programmed for a computer. The following values were calculated at each of the time step $\Delta t = \min\,[0.001;\ 1/(20f_5)]$: deflection $v(l/2,t)/v_0$, bending moment $M(l/2,t)/M_0$ and vertical acceleration $a(l/2,t)/g$ at the beam center $x = l/2$, where $g = 9.81\ \mathrm{m/s^2}$ is the gravitational constant. The calculation stops when the train (i.e. the N-th force) leaves the bridge and the beam absorbs 20 additional cycles of the free damped vibration, i.e. $T_N + 20/f_1$. The five modes of natural vibration were considered (for $x = l/2 \to j = 1, 3, 5$).

The parametric study assumed both the concrete and steel bridges of spans 5, 10, 15, 20, 30 and 50 m with appropriate first natural frequencies f_1, logarithmic decrements of damping ϑ and weights $G = \mu g l$. Their numerical values, appearing in the Table 1, were taken from the empirical formulas published in [3].

Three assumed types of high speed trains, i.e. ICE 2, Eurostar/TGV and Talgo AV 2, used in Germany, France/England and Spain, respectively, differ by their predominant forces F, their total numbers N, predominant car lengths d and total lengths. The Table 2 shows the assumed train parameters.

The time histories were calculated for all the bridge and train parameters at eleven speeds: 5, 50, 100, 150, 200, 250, 300, 350, 400, 450 and 500 km/h. Altogether $2 \times 6 \times 3 \times 11 = 396$ cases were analysed.

Table 2. Train parameters.

Characteristics	Notation	Unit	ICE 2	Eurostar/TGV	Talgo AV 2
Axle force	F	kN	195/112	170	170
Number of axle forces	N	1	56	48	40
Predominant car length	d	m	27.3	18.7	13.4
Length of the train		m	362.1	386.5	356.05

The examples in Figures 3 and 4 show from above: deflection-time history, stress spectrum (for details see the Section 5) and vertical acceleration-time history for a steel bridge of the span $l = 20$ m under the train Eurostar at the speed $c = 5$ and 500 km/h, respectively. The effect of the speed $c = 5$ km/h is assumed as the quasistatic one and the dynamic impact factors φ_v for the deflection are related to the basic speed of 5 km/h

$$\varphi_v = \frac{\max v(l/2, t) \text{ at speed } c}{\max v(l/2, t) \text{ at speed } 5\,\text{km/h}}. \tag{28}$$

The analogous definitions are true for the bending moment impact factor φ_M and vertical accelerations $a(l/2, t)/g$. The dynamic impact factor (28) cannot be confused with that one applied to the standard load model UIC 71.

The record in the Figure 1 is similar to the acceleration-time history in the Figure 4.

4 PARAMETRIC STUDY

From the comprehensive study, only the effect of some important parameters is shown here:

4.1 *Train speed*

The speed of the train movement is the most important parameter for the design of high speed lines, [5]. Figure 5 shows the effect of the speed on the dynamic impact factor of deflection v and bending moment M for three investigated trains on a steel bridge of span $l = 5$ m. The Excel programme smoothed the calculated values in the Figure 5 and in analogous drawings which follow.

All the figures of that kind indicate an increasing tendency of the dynamic impact factor with increasing speed. Besides, some local peaks appear on analogous figures that depend on the complex dynamic system bridge + train, however, the general rising inclination cannot be overlooked.

4.2 *Bridge span*

The effect of the bridge span appears on the Figure 6, valid for steel bridges under various trains at the speed $c = 350$ km/h. No general tendency can be drawn from the analogous figures, [5].

4.3 *Various high speed trains*

Figure 7 shows the dynamic effects of various trains. Their highest dynamic impact factors arrive on various spans so that it cannot be generalized for all cases, [5]. It must be pointed out that the concrete and steel bridges of the same span form two different dynamic systems as they provide different natural frequencies, damping characteristics and masses.

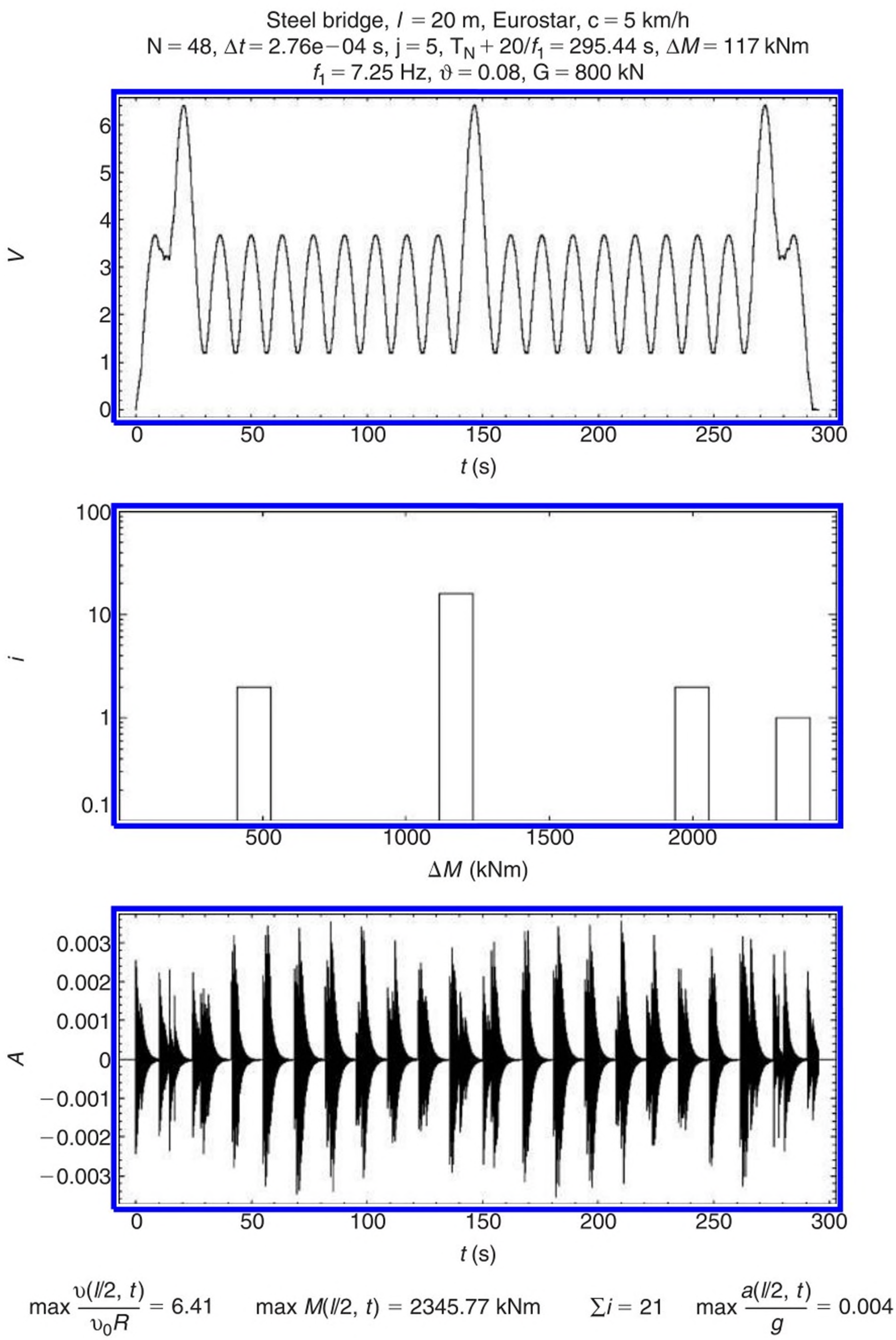

Figure 3. Steel bridge, $l = 20$ m, Eurostar, $c = 5$ km/h.

4.4 *Vertical acceleration of bridges*

The speed affects the vertical acceleration of bridges in a similar way as the deflection or bending moment, [6]. Figure 8 indicates the phenomenon as the function of the speed c for three assumed trains where the increasing tendency can be clearly seen.

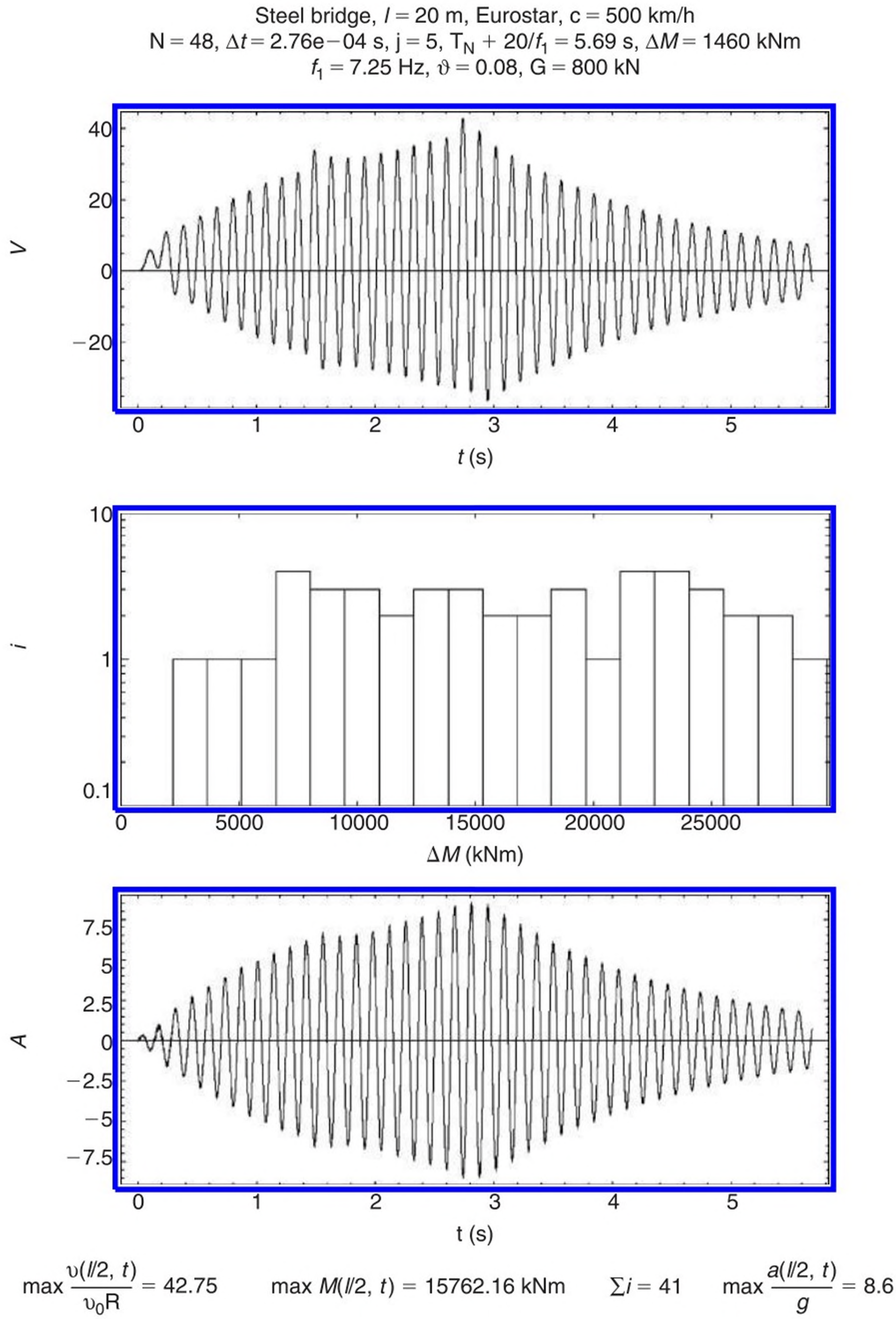

Figure 4. Steel bridge, $l = 20$ m, Eurostar, $c = 500$ km/h.

The increasing span l diminishes the vertical accelerations of bridges (Fig. 9). It is affected by the increasing mass of bridges for longer spans. The same tendency is observed for the damping (Fig. 10) and weight of bridges (Fig. 11). It must be pointed out that the resonance substantially affects the vertical acceleration of bridges as can be seen in Figures 10 and 11. The last two figures serve only for qualitative comparison.

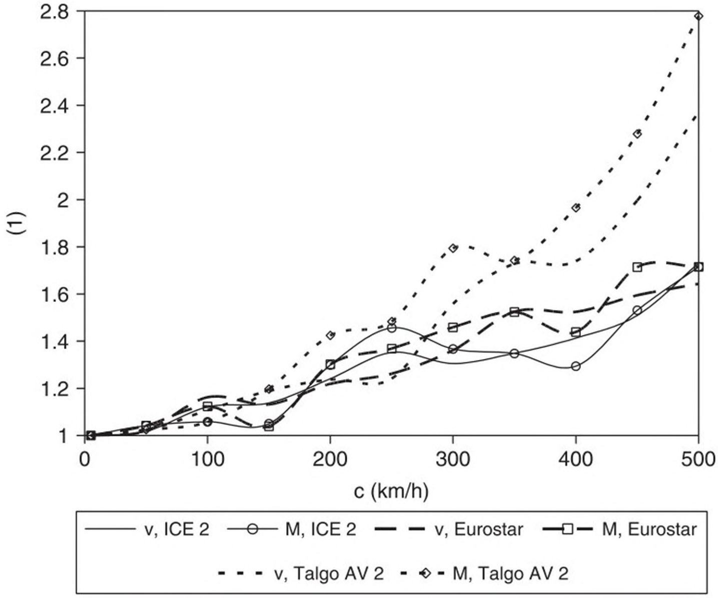

Figure 5. Dynamic impact factor as a function of the speed c for deflection v, bending moment M and various trains. Steel bridge, $l = 5$ m.

The peaks of vertical accelerations may cross the limit value for ballast (3.5 m/s^2 – see [2] and [7]). They strongly depend on damping and mass of a bridge whereby the higher the mass and the damping of bridges, the lower dynamic effects of a train. It is distinct especially at resonance. Otherwise, the vertical accelerations of bridges may become a new limit state for structures on high speed lines.

5 STRESS SPECTRA

According to the design standards (e.g. Eurocodes 1, 2 and 3), the dynamic stresses are covered by the dynamic impact factor

$$\varphi = \frac{\sigma_{dyn}}{\sigma_{stat}}, \tag{29}$$

where σ_{dyn} and σ_{stat} are the maximum dynamic and static stresses, respectively, see the Figure 12. The dynamic impact factors are prescribed by Eurocodes for various cases and speeds and serve for the strength assessment.

On the other hand, the fatigue is substantially affected by the stress ranges, [3]:

$$\Delta\sigma = \sigma_{\max} - \sigma_{\min} \tag{30}$$

defined as the difference between the local maximum $\sigma_{\max}$ and local minimum $\sigma_{\min}$, [8], [9]. Figure 12 and Eqs (29) and (30) clearly show the important difference between φ and $\Delta\sigma$, where the last value considers also the free vibration when the train left the bridge.

Knowing the section modulus W, the stresses are

$$\sigma(x,t) = \frac{M(x,t)}{W}. \tag{31}$$

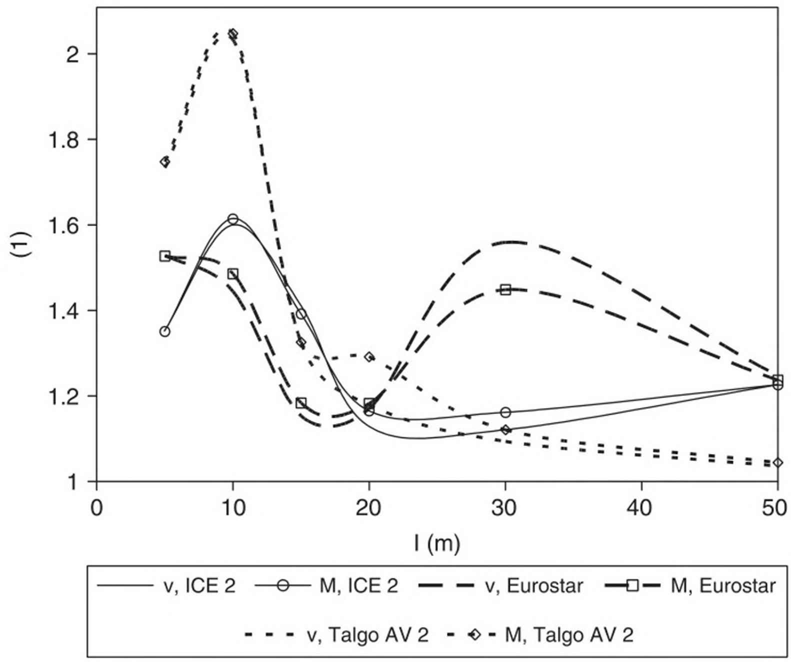

Figure 6. Dynamic impact factor of steel bridges of various spans l under high speed trains at $c = 350$ km/h, $v =$ deflection, $M =$ bending moment.

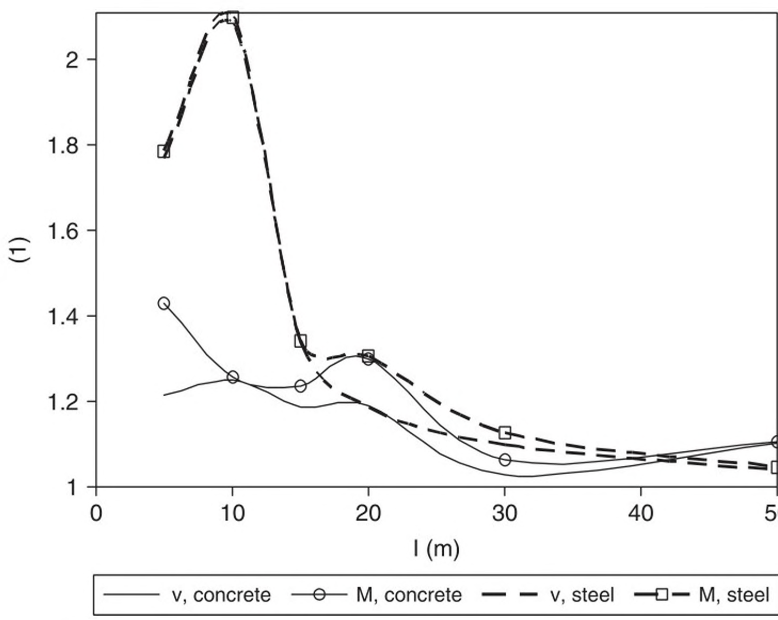

Figure 7. Dynamic impact factors of concrete and steel bridges of various spans l under Talgo AV 2 at $c = 350$ km/h.

Respecting (31) and for easing the calculations, we call "stress spectrum" instead of more exact "bending moment spectrum" in what follows. The consideration of bending moments is more general than stresses because it is not necessary to know the particular beam cross section.

The problem, how to receive the stress ranges from stress records clarify the counting methods. One of the most used is the rain flow counting method, [3], the principle of which is sketched in

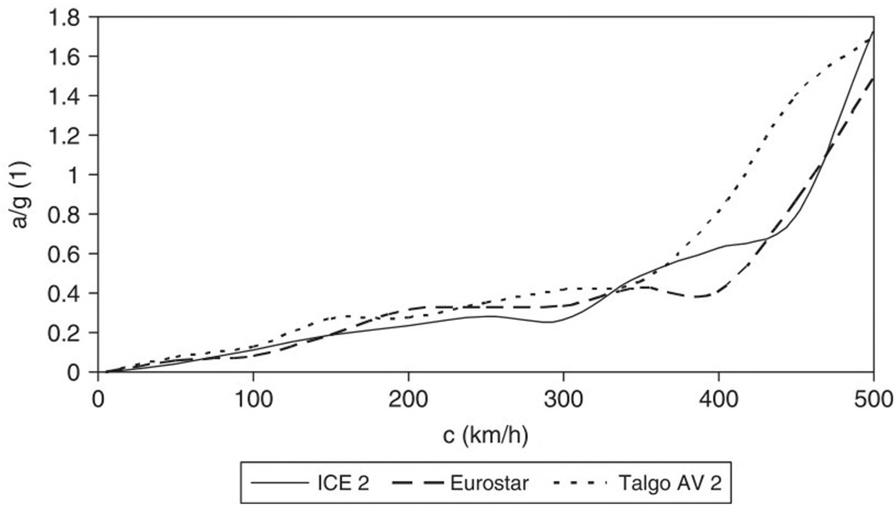

Figure 8. Vertical acceleration of a concrete bridge of span $l = 10$ m under various trains as a function of the speed c.

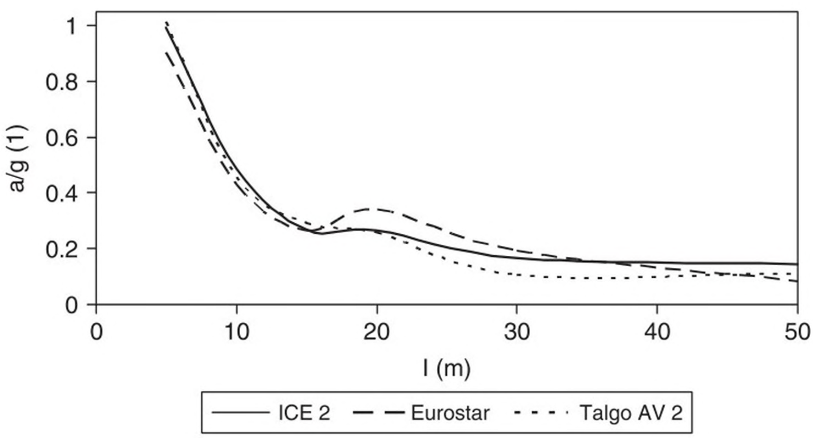

Figure 9. Vertical acceleration of concrete bridges under high speed trains at $c = 350$ km/h as a function of span l.

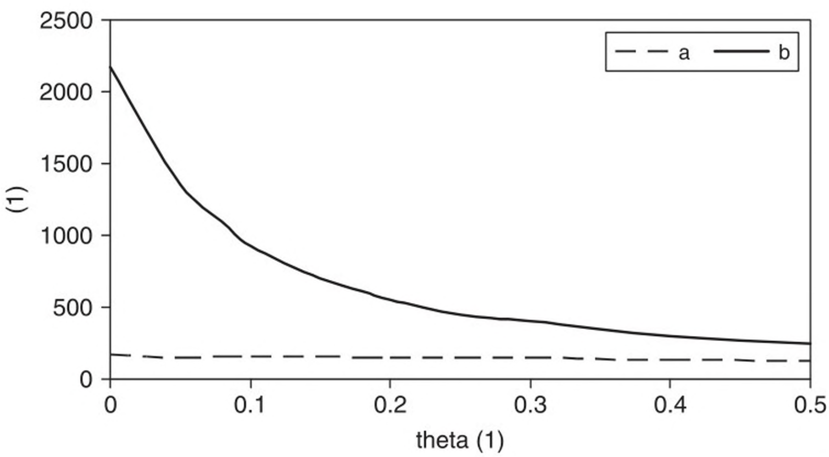

Figure 10. Effect of damping ϑ (theta) on acceleration of bridges: a) concrete bridge, $l = 20$ m, ICE 2, $c = 350$ km/h, out of resonance; b) steel bridges, $l = 20$ m, Eurostar, $c = 500$ km/h, at resonance.

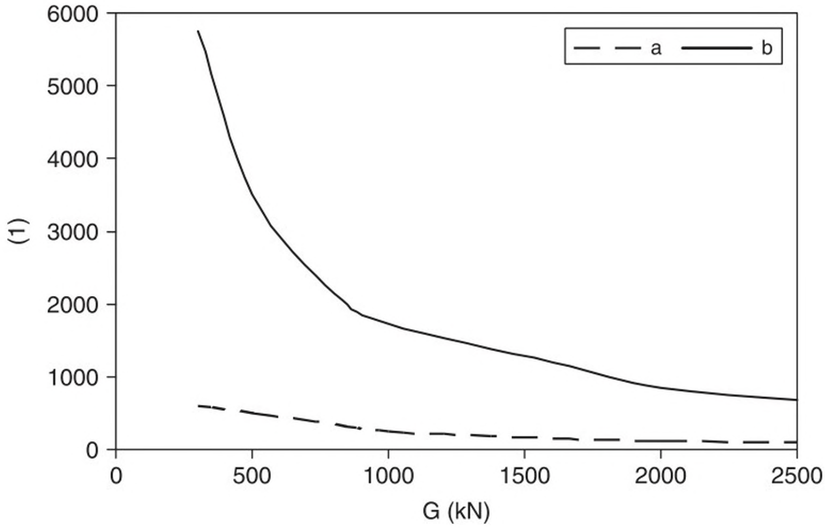

Figure 11. Effect of the weight G of bridges on their vertical acceleration: a) concrete bridge, $l = 20$ m, ICE 2, $c = 350$ km/h, out of resonance; b) steel bridge, $l = 20$ m, Eurostar, $c = 500$ km/h, at resonance.

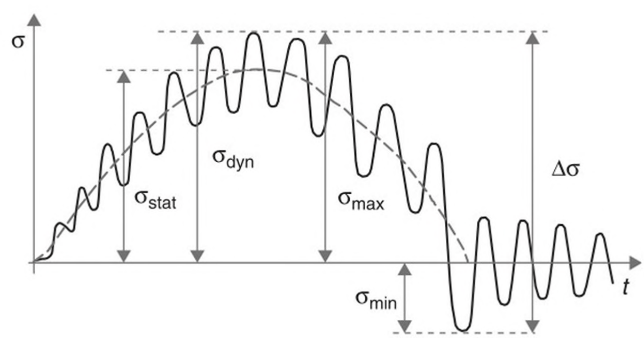

Figure 12. Stress-time history in a bridge under moving vehicles. Scheme for the calculation of the dynamic impact factor and stress range.

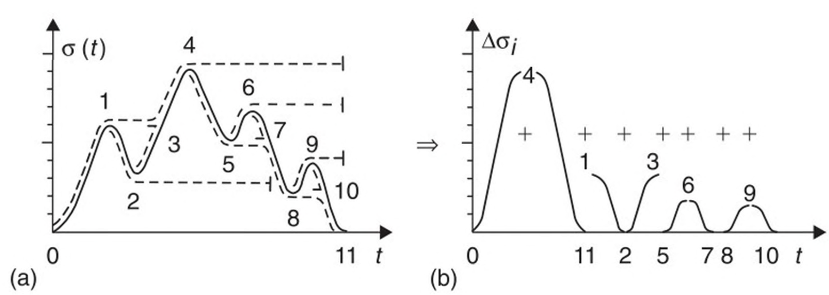

Figure 13. Rain flow counting method: a) stress-time history, b) decomposition into stress ranges.

Figure 13. The basic assumption of this method is: the fatigue damage due to small stress cycles may be added to the fatigue damage due to large stress cycles. This results in classified stress ranges which are paired into complete stress cycles.

The procedure is described in details in [3] and the classification of the bending moment time histories (23) into stress ranges (30) follows the conditions

$$A(i-1) \leq A(i+1) < A(i) \leq A(i+2), \quad or \tag{32}$$

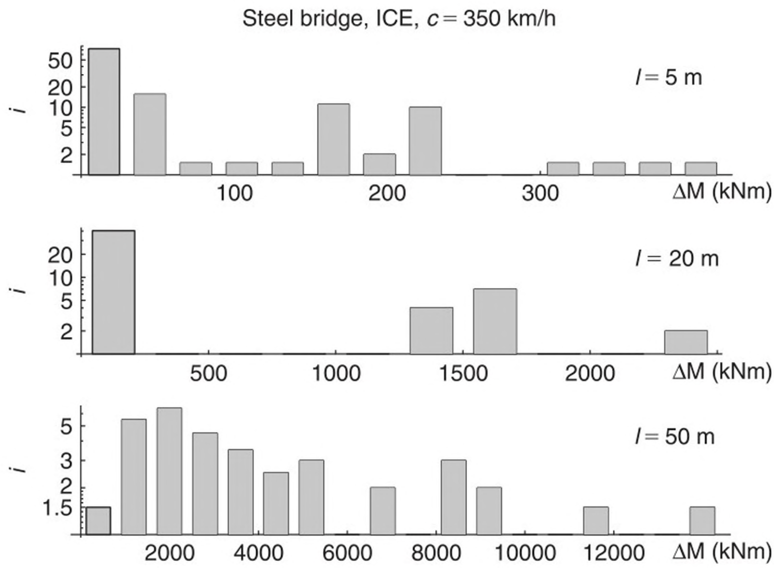

Figure 14. Stress spectra of steel bridges of spans $l = 5$, 20 and 50 m under the train ICE 2 at speed $c = 350$ km/h.

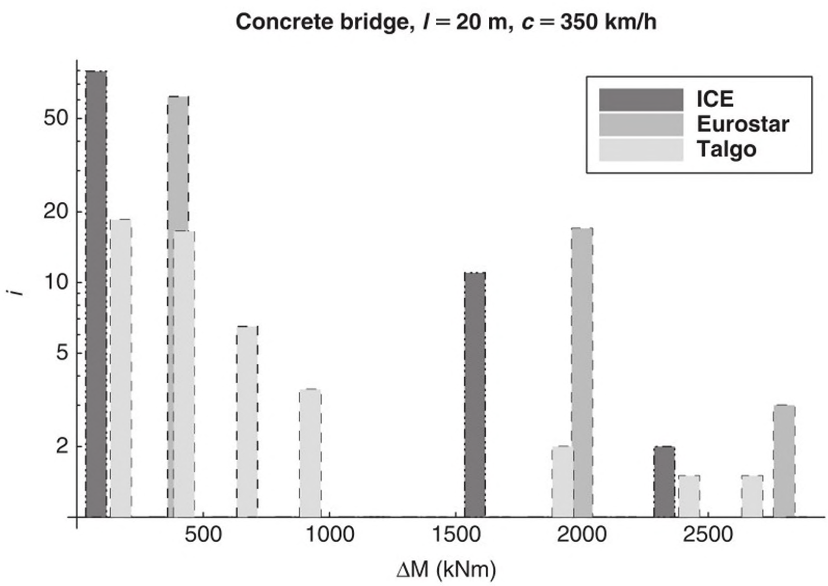

Figure 15. Stress spectra of a concrete bridge of span $l = 20$ m under various trains at speed $c = 350$ km/h.

$$A(i-1) \geq A(i+1) > A(i) \geq A(i+2), \tag{33}$$

where $A(i)$ denotes the i-th local peak. The classification provides the number i of stress cycles in each of the bending moment class ΔM (kNm) and is depicted in histograms or tables, see examples in Figures 14 and 15.

Twenty classes of bending moments were assumed under each train and speed, however, the stress ranges lower than one tenth of the basic stress range were neglected to omit the unimportant small amplitudes. The counting of negative peaks (e.g. at mid span of a simple beam, Fig. 12) explains the large stress ranges at resonant vibration.

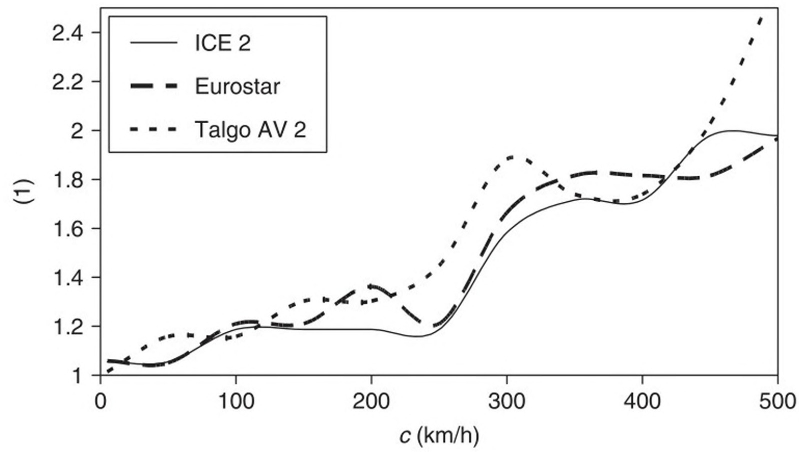

Figure 16. The maximum stress range as a function of the speed c (related to that one at the quasistatic speed of 5 km/h), concrete bridge of span $l = 5$ m under various trains.

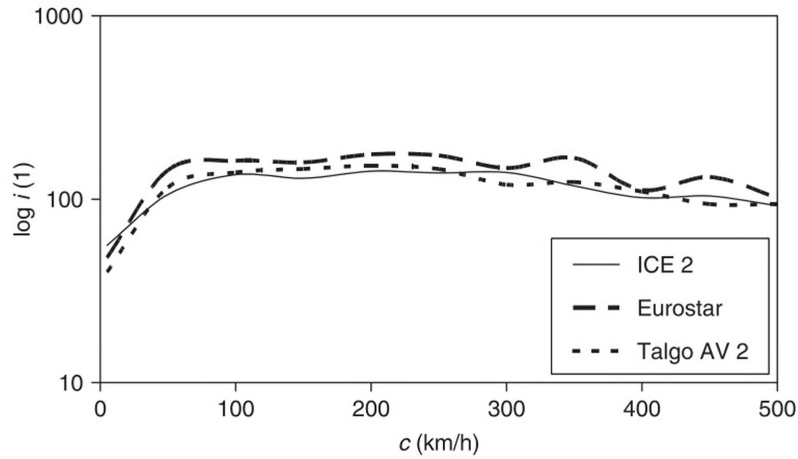

Figure 17. The total number of stress cycles i as a function of the speed c, concrete bridge of span $l = 5$ m under various trains.

The stress spectra in various steel bridges of spans 5, 20 and 50 m under the train ICE 2 at speed 350 km/h are depicted in the Figure 14. Figure 15 compares the effect of three investigated high speed trains.

The maximum stress ranges substantially depend on the speed c as depicted in the Figure 16, while the total number of stress cycles remains approximately constant, Figure 17.

6 CRITICAL SPEEDS

The analysis of the Eq. (18) shows two reasons of the intensive vibration of a beam, [11]:

1. If the forces F_n are arranged in equidistances d, then their repeated action excites the resonant vibration. It happens after a long time when the time necessary for crossing the distance d at speed c is equal to the k-multiple of the period of natural vibration $1/f_j$. The condition

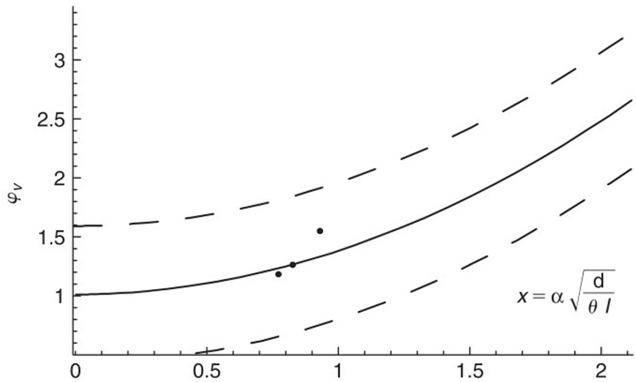

Figure 18. The dynamic impact factor φv of deflection as a function of parameters, (36), full line – mean value, dashed lines – bounds with 95% of reliability.

yields the critical speed

$$c_{cr} = \frac{df_j}{k}, \quad j = 1, 2, 3, \ldots, \quad k = 1, 2, 3, \ldots, 1/2, 1/3, 1/4, \ldots \tag{34}$$

The present high speed trains reach the speed (34), see [10], however, the resonance is disturbed by non-equal distances, groups of axle forces, etc. Therefore, the resonance is rather a sporadic phenomenon but in spite of them was observed.

2. The beam may lose its stability if the denominator D (21) in (19) and (18) tends to zero. It may happen at low damping, in limit for $\omega_d \to 0$ and $\omega_j^2 = j^2\omega^2$. Thus, the critical speed follows from (16) and (17):

$$c_{cr} = \frac{2lf_j}{j}, \quad j = 1, 2, 3, \ldots, \tag{35}$$

The second case is analogous to the stability problems. The present high speed trains have not yet reached the speed (35) as shown in [10].

7 SIMPLIFIED DESIGN OF BRIDGES

From several possibilities how to simplify the design of bridges on high speed lines, we suggest two ways.

7.1 *Statistical evaluation*

If we put together the maximum values of all investigated cases (Fig. 18, [11]) and evaluate them using the statistical methods, an empirical formula arises for the dynamic impact factor of deflections together with the reliability bound of 95%

$$\varphi_v = 1 + 0.365 \frac{\alpha^2 d}{\vartheta l} \pm 0.579. \tag{36}$$

The same procedure presents the analogous values for stresses (bending moments) and vertical acceleration, respectively:

$$\varphi_M = 1 + 0.378 \frac{\alpha^2 d}{\vartheta l} \pm 0.661, \tag{37}$$

$$\frac{a}{g} = 1.403\frac{\alpha^2 d}{\vartheta l}\frac{F}{G} \pm 1.449. \tag{38}$$

The following notations have been used in the above equations:

$$\alpha = \frac{c}{2f_1 l} \tag{39}$$

– the dimensionless speed parameter, [3], [4],
d – length of predominant cars with the axle force F,

$$\vartheta = 2\pi\zeta/100,$$

ζ – damping ratio in %.

7.2 *Estimation of maximum values*

If we calculate the same case as in the Section 2, however, as the movement of an infinite row of axle forces F in equidistances d, then the forced stable vibration of a beam presents at $t \to \infty$ the following estimations of the vibration amplitudes transformed in the dynamic impact factors, [10]:

$$\varphi_v = \frac{4\alpha^2\beta l}{\vartheta l}, \tag{40}$$

$$\varphi_M = \frac{4\alpha^2\beta l}{\vartheta d}, \tag{41}$$

$$\frac{a}{g} = \frac{8\alpha^2\beta l}{\vartheta d}\frac{F}{G}, \tag{42}$$

where

$$\beta = \frac{1-\vartheta(4\alpha)}{1-\alpha^2}$$

The formulas (40) to (42) are conservative, while the Eqs (36) to (38) seem to be more realistic. The simple formulas suggested above may serve for the first, rough and quick assessment of bridges.

8 INTEROPERABILITY

Interoperability is a technical expression which has been used by many professions in recent time. However, its definition is a little vague. The bridge engineers understand by interoperability the capability of a bridge to carry a particular train or vehicle running at a certain speed. Of course, the condition must be fulfilled on international lines without respect to the borders.

The experiments, [2], [7], have demonstrated that the critical phenomenon on high speed lines is the destabilization of ballast on bridges affected by their vertical accelerations. The maximally acceptable (ultimate) values of the bridge deck accelerations were found as

$$a_{ult} = 3.5\,\mathrm{m/s^2} \quad \text{or} \quad a_{ult} = 5\,\mathrm{m/s^2} \tag{43}$$

for bridges with ballast or without ballast, respectively, [2], [7]. Therefore, the resulting maximum acceleration (42) must be lower or equal to the value (43)

$$a_{ult} \geq a = \frac{8\alpha^2\beta l}{\vartheta d}\frac{F}{G}g. \tag{44}$$

The condition (44) could serve as a quantification of the interoperability. Besides, some other definitions of the bridge and vehicle interoperabilities exist, [10], which enable the international transport.

9 CONCLUSIONS

The high speed trains substantially affect the dynamic behaviour of railway bridges which could be brought even to the resonant vibration. It is caused by a long sequence of axle forces or their groups distributed in almost regular distances. The dynamic increments of the bridge deflection, stresses and vertical accelerations roughly raise with increasing speed. The function provides some local peaks, the positions and amplitudes of which depend on the complex dynamic interaction of the bridge with the moving train.

The train was assumed in the first approximation as a system of axle forces and should be improved in the future. Nevertheless, the idealization corresponds to the design philosophy prescribed by Eurocodes and probably presents the conservative values. Of course, the load model cannot clear all the details of the problem. The derived approximate formulas may serve to the first, rough and quick assessment of bridges.

Even the presented simple theoretical model confirms the possibility of resonant vibration of bridges which appears at speeds higher than 200 km/h, but it is often disturbed by irregularities of axle distances, short duration of the train run and damping.

The dynamic response of steel bridges is higher than of concrete ones because the mass and damping of concrete are greater than of steel. The relative dynamic increments of stresses are a little higher than that of deflections. The damping substantially affects the highest peaks at resonance while outside the resonant conditions it slightly diminishes the amplitudes.

The vertical accelerations of short and medium span bridges attain considerable amounts and may cross the ultimate values. It could be dangerous for the ballast destabilization. The accelerations are diminishing with increasing span, mass and damping of bridges and are lower on concrete than on steel bridges. The routine design of bridges has not yet assumed with the acceleration of bridges. Therefore, the vertical acceleration of bridges will probably become a new limit state for the design of high speed lines in the future.

The stress spectra change their form with the increasing speed of trains: they are poor at low speeds but the high speeds substantially enlarge the stress cycles. On the other hand, their number remains almost constant. In particular, it is noticeable at resonance, where the counting methods record even the negative peaks which appear at the free vibration after the train left the bridge.

To restrict the vibration of bridges, the development of active and passive dampers for both the bridges and vehicles is recommended.

ACKNOWLEDGEMENT

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CHAPTER 9

Dynamic analysis of hyperestatic structures under high speed train loads

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ABSTRACT: The construction of new high speed railway infrastructure in many European countries is actually a great civil engineering effort. One of the main design issues is related to the fact that trains at speeds above 220 km/h may induce resonance effects in structures as bridges, subway frames, etc. In consequence, the design of such structures requires dynamic calculations. Several methods are available for the dynamic calculations of structures under high speed train loads. The simplest ones are based on sums of harmonic terms, providing upper bounds for the dynamic response, and with application limited to isostatic bridges. Alternatively, for statically indeterminate structures some models based on the direct time integration of the equations of motion are available. These methodologies suitable for hyperestatic structures, performed on full or reduced models, are discussed in this paper. Additionally, some recent results for high speed traffic loads on bridges are presented. The object of these studies is diverse: simplified torsion analysis, a proposal of a simplified method for dynamic analysis of portal frames, and several practical results obtained for concrete bridges, composite steel-concrete bridges and pre-casted concrete bridges.

1 INTRODUCTION AND STATE OF THE ART

The construction of new transport infrastructure constitutes since the last years one of the major civil engineering efforts in many European countries. In Spain, the main part of these inversions is dedicated to funding the construction of new railway lines for high speed trains, being this item the most important too in other countries as France. These new railway lines are a very competitive alternative to other transport modes for medium distances. At this moment in Spain there are several high speed railways in operation: Madrid–Sevilla, Córdoba-Málaga and Madrid–Tarragona, pertaining the last one to the railway line Madrid–Barcelona–France. The ADIF authority have in project or construction several railway lines as Madrid–Valencia–Murcia and Madrid–Segovia–Valladolid, being this one considered in the frame of the international high speed railway system Portugal–Spain–Rest of Europe.

These activities have remarked out an important engineering aspect joined specifically to the design of high speed railway structures: the dynamical effects associated to the train moving loads, for which basic solutions have been described by Timoshenko and Young [18] and discussed fully in [6,7]. Also remarkable are the contributions performed in Spain by Alarcón [1,2].

Most engineering design codes for railway bridges have followed the approach of the dynamic factor proposed by UIC [19], which takes into account the dynamic effect of a single moving load and yields a maximum dynamic increment of $\varphi' = 132\%$ for an ideal track without irregularities. The irregularities are taking into account with another parameter φ'', leading to the dynamic factor $\Phi = 1 + \varphi' + \varphi''$. This approach cover the dynamic effects associated to a single moving load but does not include the possibility of resonant effects due to the periodicity of the moving loads, as this phenomenon does not appear for train speeds below 200 km/h. In this fact, for common structural eigenfrequencies and the distance between the axles in real trains, resonance is all too real for high speed railways, and its effects may surpass largely that of a single moving load.

Really, for velocities upper than 200 or 220 km/h and distances of axles between 13 m and 20 m. resonant effects may appear. An illustrative example showing experimental resonant measurements and the corresponding computational results is showed in [4] for the Spanish AVE train crossing at 219 km/h the Tagus bridge.

New European codes include the need for dynamic calculations covering resonant behaviour [5,11,14]. The calculation procedures foreseen in these codes are easy to apply to isostatic structures, as simply supported decks, because there is a fundamental eigenmode of oscillation. The simplest ones are those based on sums of harmonic terms, which provided bounds for the dynamic response [8]. Nevertheless for hyperestatic structures, like continuous deck bridges and portal frames for railway underpasses, more sophisticated methods of analysis are required because the participation of several eigenmodes in the dynamic response.

Generally these methods are based in the direct integration of the structural response for moving loads modelling the axles of the train. The calculations may be performed on full or reduced models with or without vehicle-bridge interaction. If vehicle-bridge interaction is considered the complexity of the model is increased and a major computational effort is required. Although these kind of simulation are very interesting from a research point of view they are not useful for standard design calculations except for unusual situations. Anyway, the structural model may be analysed either by the complete discretised system with N degrees of freedom and a time integrator like the Newmark method, or by a prior modal analysis and a posteriori time integration of the n significant eigenmodes.

2 MODELS BASED ON DIRECT INTEGRATION WITH MOVING LOADS

These class of methods are based on the direct time integration of the dynamic equations of the structure, under the actions corresponding to a train of moving loads of fixed values which values are representative of each axle of the train. The structural model may be analysed through the integration of the complete discrete system with N degrees of freedom, or through a reduction of the number of degrees of freedom via a previous modal analysis which reduces substantially the number of equations. This modal analysis can be performed through an approximate numerical procedure in order to obtain de eigenfrequencies and eigenmodes of vibration. This kind of procedures are available in the majority of finite element codes, and for certain cases of simple structures the spectral analysis can be achieved by analytical methods.

Finite element methods are applicable generally to arbitrary structures, even when non linear effects must be considered. A spatial semidiscretisation of the structure is performed into subdomains called "finite elements" leading a discrete N-d.o.f. system of equations:

$$M\ddot{d} + C\dot{d} + Kd = f(t) \tag{1}$$

where $\mathbf{M}$ is the mass matrix, $\mathbf{C}$ is the damping matrix, $\mathbf{K}$ is the stiffness matrix, $\boldsymbol{f}(t)$ is the vector of external loads, and $\boldsymbol{d}$ is the unknown vector of nodal displacements. In order to integrate in time this system of equations, a direct integration of the model solves the complete system (1) for each time step, and because the equations are generally coupled they must be solved simultaneously. This procedure is valid even when nonlinear effects must be taken into account. In such case the elastic internal forces and viscous damping should be replaced by a general nonlinear term $F^{int}(d, \dot{d})$.

If as usual the structural behaviour is linear, a modal analysis can be performed resulting in another system with a remarkable reduction of degrees of freedom. In a first stage, the eigenvalue problem corresponding to the undamped system is performed solving the generalised eigenproblem corresponding to the structural discrete system:

$$(-\omega^2\mathbf{M} + \mathbf{K})\,\boldsymbol{a} = 0 \tag{2}$$

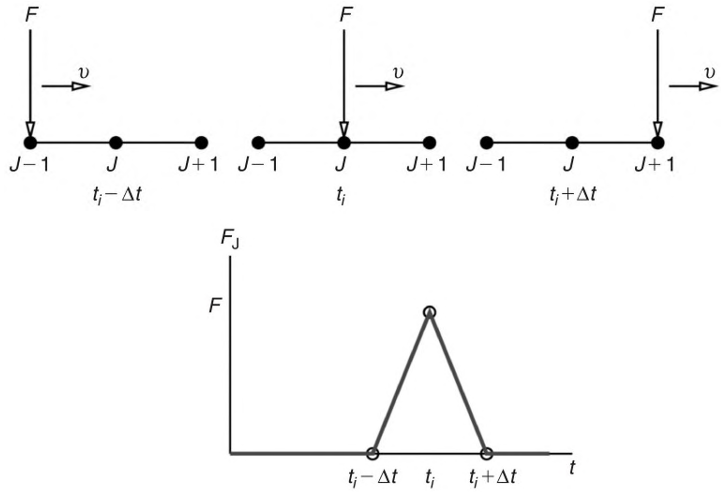

Figure 1. Nodal force time history definition for an axle load F moving at velocity v.

obtaining the more significant n eigenfrequencies ω_i $i = 1, \ldots, n$, and the corresponding normal vibration modes $\boldsymbol{a}_n$, being generally $n \ll N$. In a second stage, the vibration modes $\boldsymbol{a}_i$ oscillating with their respective frequencies ω_i are integrated in time. With this procedure the equations are decouple, and the modal response of each mode is obtained from the dynamic equation of a system with a single degree of freedom.

The simplest procedure to define the train loads is applying load histories in each node. For time step t_i and an axle load F, a nodal load F_J is assigned to the node J if the axle is above an element that contains node J. The magnitude of F_J depends linearly on the distance from the axle to the node. This procedure is outlined in Figure 1 for a single load. This scheme has been implemented in the finite element program FEAP [17] both for the ten HSLM-A (High Speed Load Model) trains defined in the Eurocode [11,14] and for the seven real trains defined in the Spanish code IAPF [14]. All the results described in this paper have been obtained with the methodology described in this section, using a direct time integration of the significant eigenmodes.

3 DYNAMIC RESPONSE OF PORTAL FRAMES

3.1 *Analytical methods*

Although for an isostatic structure the eigenfrequencies and the eigenmodes can be obtained in analytical closed form [3], it is not possible in general to perform an analytical extraction of frequencies and vibration modes for statically redundant structures. Nevertheless, is some cases a closed form solution may be obtained such as intraslational portal frames and continuous beams with two or three spans [7]. Anyway, when possible, the dynamical calculations in closed form for hyperestatic structures are more complex than those available for simple supported beams. For example in the procedure corresponding to portal frames [13], the expression of the frequency for the first eigenmode (see Fig. 2) is:

$$\omega_1 = \left(\frac{b}{l_d}\right)\sqrt{\frac{E_d I_d}{\overline{m}_d}} \tag{3}$$

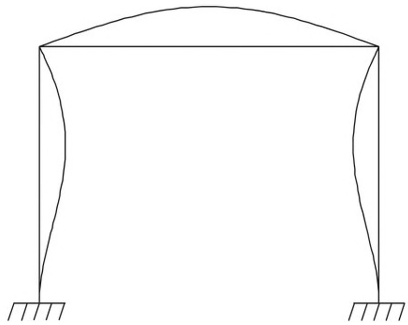

Figure 2. First vibration mode of a underpass modelled as a portal frame.

where l_d is the span of the deck, $E_d I_d$ its bending stiffness and $\overline{m}_d$ its mass per unit length. The parameter b is the solution of the following non-linear equation:

$$\frac{k_p(1-\cosh(k_i b)\cos(k_i b))}{\cosh(k_i b)\sin(k_i b)-\sinh(k_i b)\cos(k_i b)}+\frac{1-\cosh b\cos b}{(\cosh b+1)\sin b-(\cos b+1)\sinh b}=0 \quad (4)$$

where:

$$k_p=\sqrt[4]{\frac{I_d^3}{I_h^3}\frac{\overline{m}_d}{\overline{m}_h}}, \quad k_i=\frac{l_h}{l_d}\sqrt[4]{\frac{I_d}{I_h}\frac{\overline{m}_h}{\overline{m}_d}} \quad (5)$$

and being l_h, I_h and $\overline{m}_h$ the height, moment of inertia and mass per unit length of the vertical walls of the portal frame.

Once the vibration modes are known, it is necessary to integrate the dynamic equations. To this end, the basic solution is the response to a single moving load (see Fig. 3). The differential equation for a load crossing the portal deck at a constant speed v, considering the i-th mode, is:

$$M_i\ddot{y}_i+2\xi_i\omega_i M_i\dot{y}_i+\Omega_i^2 M_i y_i=F\langle\phi_i(vt)\rangle \quad (6)$$

where $\phi_i(x)$ is the modal shape, M_i the modal mass, ω_i the eigenfrequency, y_i the modal amplitude, ξ_i the critical damping fraction, and $\langle\phi(\cdot)\rangle$ is the bracket:

$$\langle\phi(x)\rangle=\begin{cases}\phi(x) & \text{if } 0<x<l_d\\ 0 & \text{otherwise}\end{cases} \quad (7)$$

After obtaining the response for a single moving load, the response for a load train may be assembled as the superposition of the responses for each one of the point loads defining the train. In this case the differential equation corresponding to mode i is:

$$M_i\ddot{y}_i+2\xi_i\omega_i M_i\dot{y}_i+\omega_i^2 M_i y_i=\sum_{j=1}^{n_{axles}} F_j\big\langle\phi_i\left(vt-d_j\right)\big\rangle \quad (8)$$

3.2 *Simplified models*

Portal frames are hyperestatic structures and therefore it is necessary a direct time integration method, including several eigenmodes, in order to evaluate their dynamic response. Besides, as they are often embedded in an embankment, they may have earth close to piers and even on the deck.

The detailed analyses of the frame considering this fact is fairly complex, being usual to consider the earth as added masses without structural effects. From a practical point of view, the conclusion is that portal frames are very simple structures, commonly encountered as railway underpasses, with low budget for calculations, but requiring a relatively great computation effort.

With this motivation the objective of the research work reported here, and detailed in a previous technical report [13], was to establish a adequate model based on an equivalent isostatic beam which dynamic response covers the corresponding one to a portal frame. This equivalent beam, with fictious mass, length and stiffness, should have similar dynamic characteristics as the frame deck, but exhibiting a dynamic envelope similar or greater than the real frame one. Four representative portal frame underpasses constructed [16] in the Spanish high speed railway line Córdoba-Málaga, with l_d deck spans equal to 8.5, 8.7, 9.8 and 15 m, have been considered.

In order to establish the most adequate equivalent beam, four beams have been considered for each frame with deck span l_d. Therefore a total of 20 beams or frames structures have been analysed. The length l_{eq} considered for each one of these equivalent beams is l_d, $0.95l_d$, $0.90l_d$ and $0.85l_d$. The mass of the equivalent beams is defined with the same value that the mass of the deck: $\overline{m}_{eq} = \overline{m}_d l_d / l_{eq}$. The equivalent beam bending stiffness $(EI)_{eq}$ is adjusted such that the first eigenfrequencies are coincident:

$$(EI)_{eq} = \frac{\omega_{frame}^2 \overline{m}_{eq} l_{eq}^4}{\pi^4} \tag{9}$$

The same damping rate has been considered for the frame and beam models, and the contribution of earth cover has been neglected. This last assumption leads to conservative results because additional masses would decrease the maximum displacements and accelerations.

The computational analyses have been carried out in accord to the methodology described in previous section, using a enhanced version of the FEAP code [17]. The calculations are performed for a range of velocities of 120–420 km/h every $\Delta v = 5$ km/h, considering the seven European high speed trains defined in the Spanish IAPF code [17]: AVE, EUROSTAR 373/1, ETR-Y, ICE-2, TALGO-AV, THALYS and VIRGIN.

The results obtained comprise displacements and accelerations. As representative examples, in Figures 4 and 5 the envelopes of maxima acceleration and displacement obtained with the TALGO AV train for the four frames considered are showed.

The criterion for selection of the most appropriate beam model was the greatest similarity between its acceleration envelope and the frame one, since this is a critical aspect in this kind of structures. Attending to this criterion, the equivalent beam of equivalent length l_d was selected. Hence the properties of the equivalent isostatic beam are $l_{eq} = l_d$, $\overline{m}_{eq} = \overline{m}_d$, damping rate ξ_{eq} as the portal frame deck, and the bending stiffness $(EI)_{eq}$ obtained from expression (9). Other remarkable conclusions obtained from this study were:

- It is possible to define an equivalent isostatic beam for the dynamic analysis of usual portal frames in railroad underpasses, which conserves the form of envelopes of accelerations, displacements and impact coefficients.
- For non critical speeds (those velocities for which the results obtained are lower than the maximum ones) the results obtained for the equivalent beam are almost always greater than the results for the portal frame. Nevertheless it cannot be stated with absolute generality that the results obtained for an equivalent beam are always an envelope for the portal frame ones.

4 EXAMPLES

4.1 *Methodology for the analysis of hyperestatic bridges*

In this section some dynamic analyses of representative real hyperestatic bridges in the Spanish high speed lines are showed. The methodology followed for the analyses, described in general

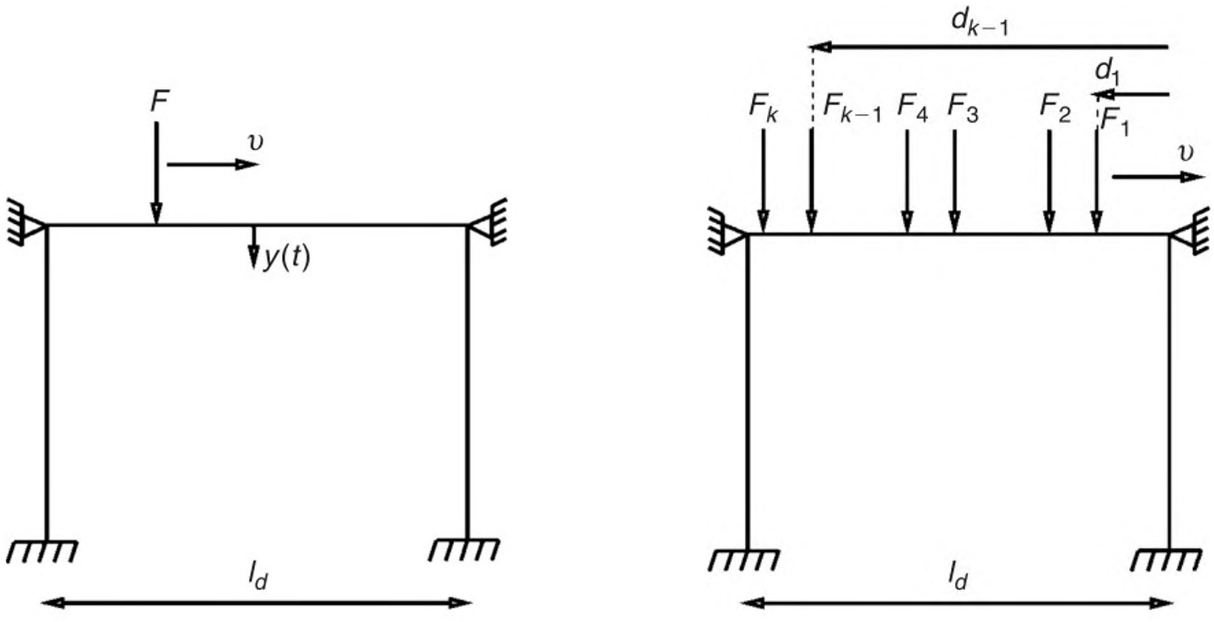

Figure 3. Response of a portal frame for a single moving load and for a load train.

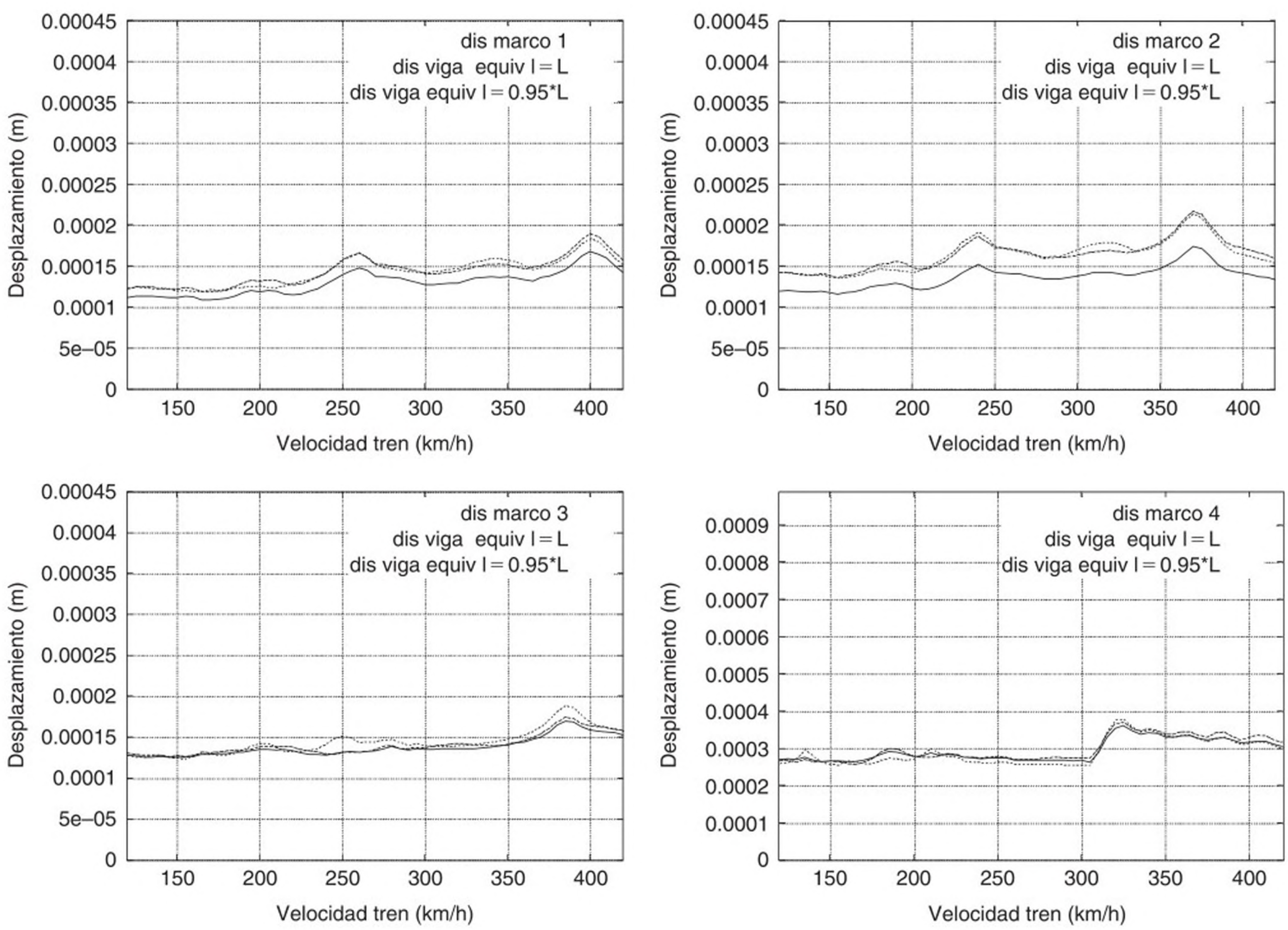

Figure 4. Envelopes of maximum displacements for portal frames and their equivalent beams of lengths l and $0.95l$ for TALGO AV train.

in section 2, is explained in this subsection in the frame of the Spanish IAPF code [14] and the Eurocode [11].

The structure is modelled with three dimensional linear frame elements based on the Euler-Bernoulli beam theory, in order to consider in a coupled way the bending and torsion effects. At a first step a modal analyses of the structure is considered in order to know the eigenfrequencies and eigenmodes of the structure. In a second step the eigenmodes up to 30 Hz (in accord to the

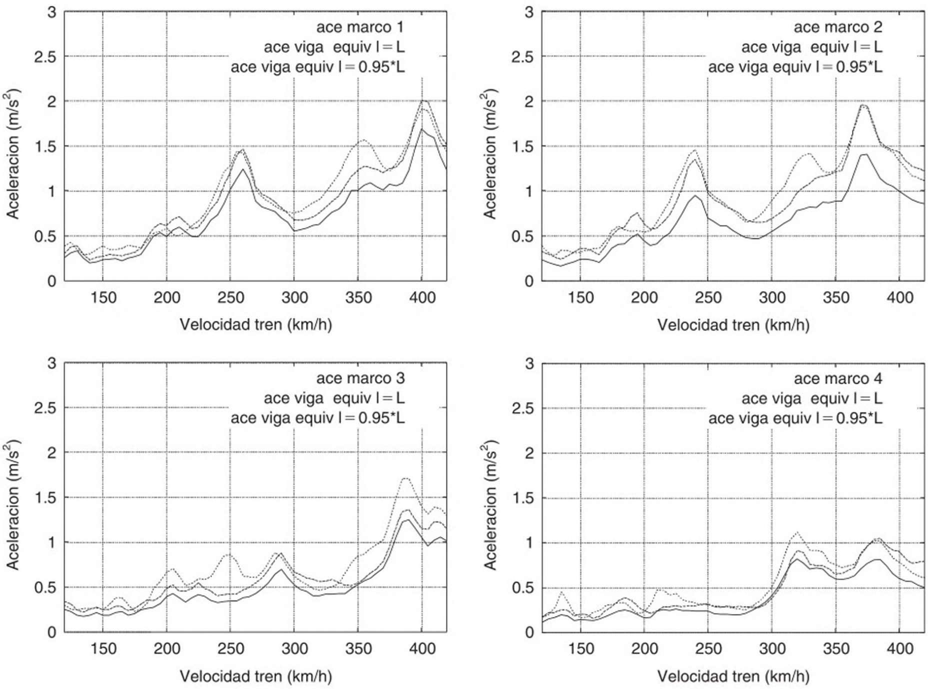

Figure 5. Envelopes of maximum accelerations for portal frames and their equivalent beams of lengths l and $0.95l$ for TALGO AV train.

Spanish IAPF code [14]) are integrated in time. The loads considered are the corresponding to the European trains defined in IAPF 2003 code [14] or, preferably in order to assess the European interoperability, the HSLM-A (High Speed Load Model) trains defined in IAPF 2003 [14] and Eurocode [11]. The calculations are performed for a range of velocities from 120 km/h to 420 km/h (the maximum velocity in the analyses is a 20% higher than the maximum velocity of the bridge design [14]). The mass per unit length of the elements is defined taking into account the proper mass of the structure plus the corresponding to the rest of the permanent loads: track, ballast, sleepers, rails, etc. The damping is defined as Rayleigh damping [3] considering a fraction of critical damping rate ξ equal to 2% for concrete structures and 0.5% for steel structures [14].

From the analyses described in the later paragraph the maximum movements and accelerations (for displacements and rotations) are obtained for the points of interest, for each velocity. In accord to the Spanish codes, the verification of the ELU (Ultimate Limit State) and ELS (Usage Limit State) states must be assured. The ELU one is verified through the definition of the impact coefficient [14]:

$$\Phi = \max_i \frac{\delta^i_{\text{din,real}}}{\delta^i_{\text{est,tipo}}} \tag{10}$$

In expression (10) the parameter $\delta^i_{\text{est,tipo}}$ is the static deflection obtained from train UIC71. The parameter $\delta^i_{\text{din,real}}$ is the "real" displacement obtained for the i train. This value is related with the ideal displacement $\delta^i_{\text{din,ideal}}$ which is obtained from the dynamic analysis considering the i train, and being the highest value for all the velocities considered. This relation is (for structures with good maintenance):

$$\delta^i_{\text{din,real}} = \delta^i_{\text{din,ideal}} \left(1 + \varphi' + 0.5\varphi''\right) \tag{11}$$

Figure 6. Río Cabra viaduct (Courtesy of NECSO).

being φ' a coefficient related to the equivalent length of the bridge [14] and to its fundamental bending frequency, and φ'' a coefficient related to the tracking irregularities [9].

In relation with the dynamic effects, the verification of the ELS state comprises the checking of several conditions [15]:

- The value of accelerations must be lower than $0.35g$ for bridges with ballast, and lower than $0.5g$ for tracks without ballast.
- The value of torsion warping between two sections with a distance equal to 3 m is limited both in the Spanish IAPF code [14] and the Eurocode [11], being more restrictive the latter one.
- The bending rotation in the end supports and the relative bending rotation in the intermediate supports (for bridges with isostatic decks) have a limited value. This value is $\theta \leq 3.5 \cdot 10^{-3}$ for the transition tie bar–deck and $\theta \leq 5 \cdot 10^{-3}$ for the transition between adjacent decks. These values are modified for tracks without ballast and for bridges with only one track [14].
- In order to avoid possible transversal resonant effects in the vehicles, the eigenfrequency corresponding to transversal bending must be higher than 1.2 Hz.
- In order to guarantee the comfort of the passengers the vertical deformation of the bridges are limited depending on the span and the speed of the train.

In the following paragraphs several results obtained from the dynamical calculations of different types of real bridges are showed.

4.2 *"In situ" concrete bridges*

The Río Cabra viaduct is in Córdoba-Málaga High Speed line. It is a continuous deck viaduct with seventeen spans and hollow slab transversal section. The length of the spans are 20 m for the end ones and 25 m for the intermediate ones, resulting in a total length of 415 m. In Figure 6 a image of this viaduct at the ending stage of construction is showed. In Figure 7 the envelopes of impact coefficient Φ, computed in accord to expression (10), and acceleration obtained from the dynamic calculations are showed, corresponding to the mid point of the side span. For the acceleration envelope the influence of the track irregularities is showed.

The data of this viaduct, together with the corresponding to one with box slab section, was used in order to analyse the simplified method proposed for torsion analysis by ERRI D214 committee [10]. The object of this study, detailed in [13], was to compare the cited simplified methodology

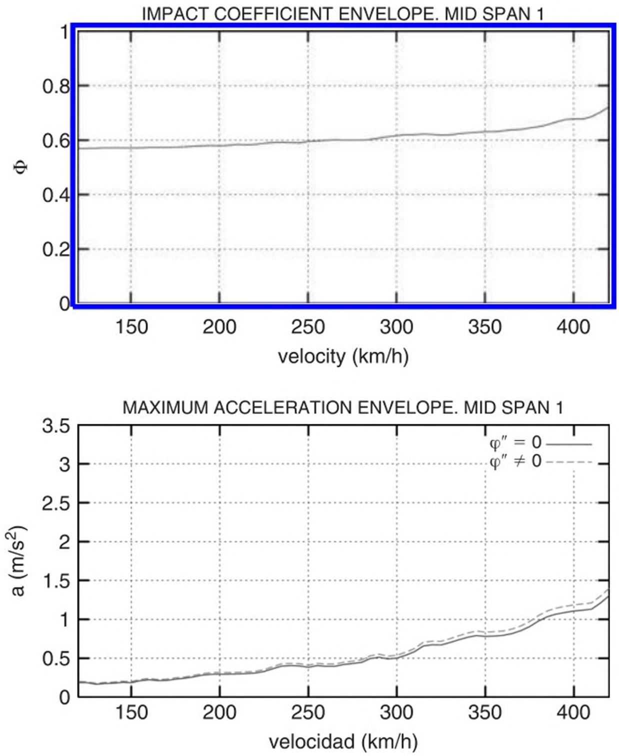

Figure 7. Impact coefficient and acceleration envelopes computed for Río Cabra viaduct.

with the application of a complete three dimensional analyses coupling the bending and torsion effects. To this end the following process was followed:

– Finite element dynamic analysis to evaluate the effects due to longitudinal bending.
– Dynamic analysis to evaluate the effects due only to torsion, obtaining them by two different ways:
  1. From a bending analysis of an isostatic beam, applying the existing proportionality between the maximum acceleration curves associated to torsion and bending.
  2. From the finite element model used for torsion results.
– Finite element complete dynamic analysis coupling bending and torsion effects.

A comparison criterion was established to relate the results obtained with the simplified method (direct sum of the absolute maximum of response obtained for the *only bending* analysis plus the maximum obtained from the *only torsion* analysis), with the maximum corresponding to the complete analysis and with the SRSS (Square Root of the Sum of Squares) combination. From this study, the following conclusions was extracted:

– The simplified method proposed in [10] is valid, but in certain situations is too conservative. The simplified method modified with the uses of SRSS combination is not always on the safety side, always the deviation with these examples was small.
– For structural sections with large torsional stiffness GJ_t, the deviation of results obtained with the simplified method and those obtained with the coupled model of bending-torsion is almost insignificant.

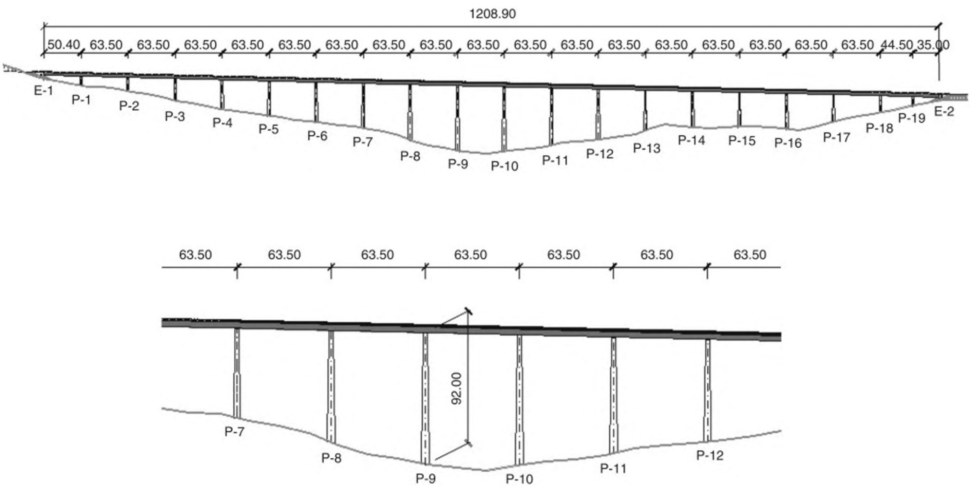

Figure 8. de Las Piedras viaduct. Longitudinal profile (Courtesy of IDEAM).

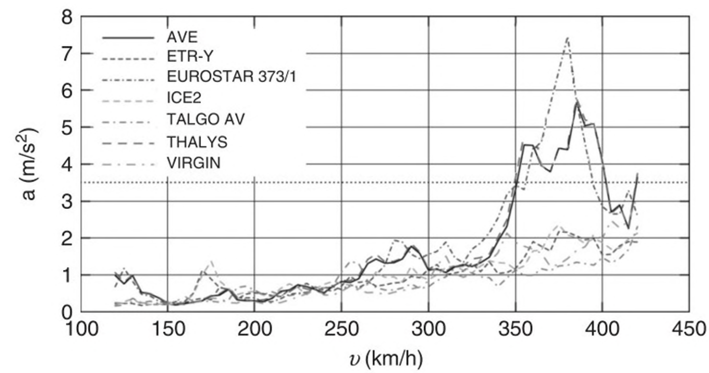

Figure 9. Envelope of maximum accelerations for open sections. Mid point of 35 m side span.

– For sections with reduced torsional stiffness (for example, hollow slabs or open sections) the deviation of results between simplified models and those of interaction bending-torsion is more significant (specially for isostatic beams).

4.3 *Concrete–steel composite bridges*

Typologies corresponding to concrete–steel composite bridges are usual in French high speed lines, but not so common in the Spanish ones. The Arroyo de las Piedras bridge, located in the Córdoba–Málaga HS line, is the first composite bridge in a Spanish high speed line. It has a total length of 1208 m composed of 20 spans. The intermediate spans are 63.5 m in length, and the side spans are 50.4 m and 35 m. In Figure 8 a longitudinal profile of the bridge is showed.

One of the main characteristics of composite bridges, from a dynamical point of view, is the low values of eigenfrequencies associated to the torsional oscillation modes. This fact leads to the participation of a high number of eigenmodes in the computations. Besides, the values for bending deflections and torsional deflections of the deck are similar being necessary to compute both effects in a coupled way.

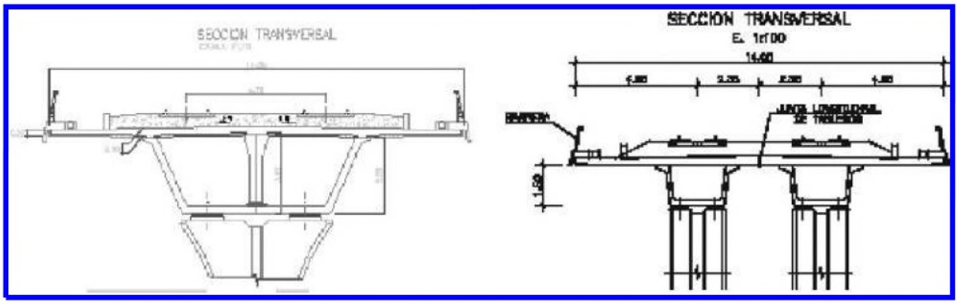

Figure 10. Attached double box and double box beam sections of precasted bridges (Courtesy of PRAINSA).

Figure 11. Río Moros viaduct (Courtesy of PRAINSA).

Figure 12. Double box section bridge (Courtesy of PRAINSA).

For the Arroyo de las Piedras viaduct several dynamic analyses have been carried out in order to analyse the influence on different design parameters. The seven real European trains defined in IAPF code [14] have been considered, for a range of velocities from 120 km/h to 420 km/h every 5 km/h. The computational analyses were performed using a modified version of FEAP code [17]. The following aspects were considered in the sensitivity analyses carried out:

1. Comparison of results obtained considering a damping ratio $\xi = 0.5\%$ (specified in [14] for composite and metallic structures calculations) with those obtained considering $\xi = 1\%$.
2. Comparison of results obtained considering encastred torsion supports and elastic torsion supports.

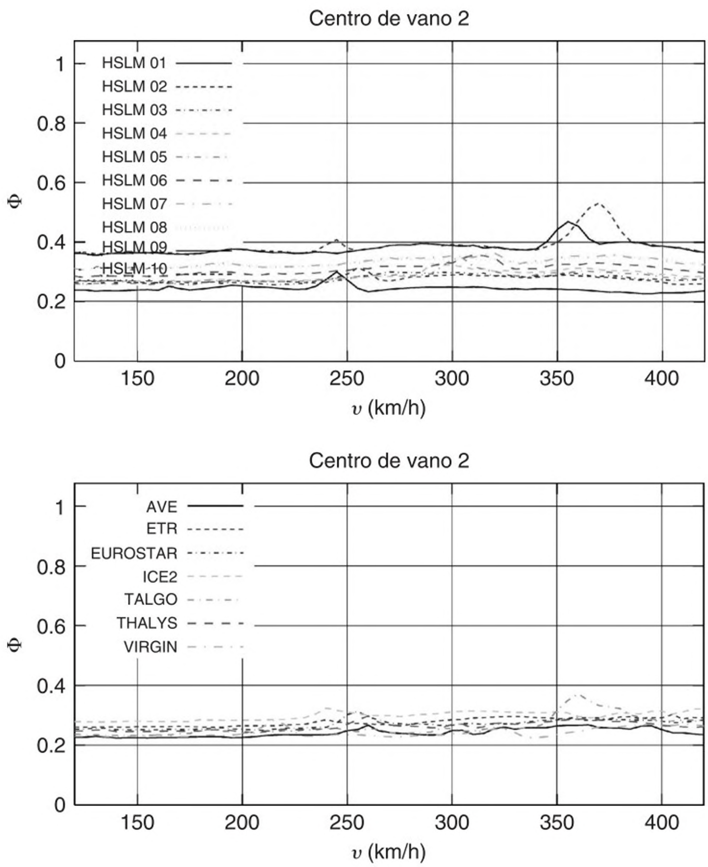

Figure 13. V2 viaduct. Impact coefficient envelopes for universal trains HSLM-A and European real trains.

3. Sensitivity to the results corresponding to the evaluation of bending inertia under the hypothesis of total cracked section in the part of the deck with negative bending moments.
4. Variation of the results obtained considering open sections for the torsional response of the structure.

The results obtained from these analyses are detailed in [12], being reported the following conclusions:

1. The maximum vertical accelerations increase a 33.3% from considering a damping fraction $\xi = 1\%$ to $\xi = 0.5\%$. For both values the Usage Limit State requirements are verified.
2. The values computed with and without encastred torsional supports are very similar.
3. The results corresponding to the hypothesis of cracked sections for negative bending moments don't show important differences with the standard ones.
4. The main differences appear considering open sections. The increment of the impact coefficient is important although it does not modify the design parameters of the viaduct. Nevertheless the values obtained for maximum accelerations are not admissible as can be seen in Figure 9.

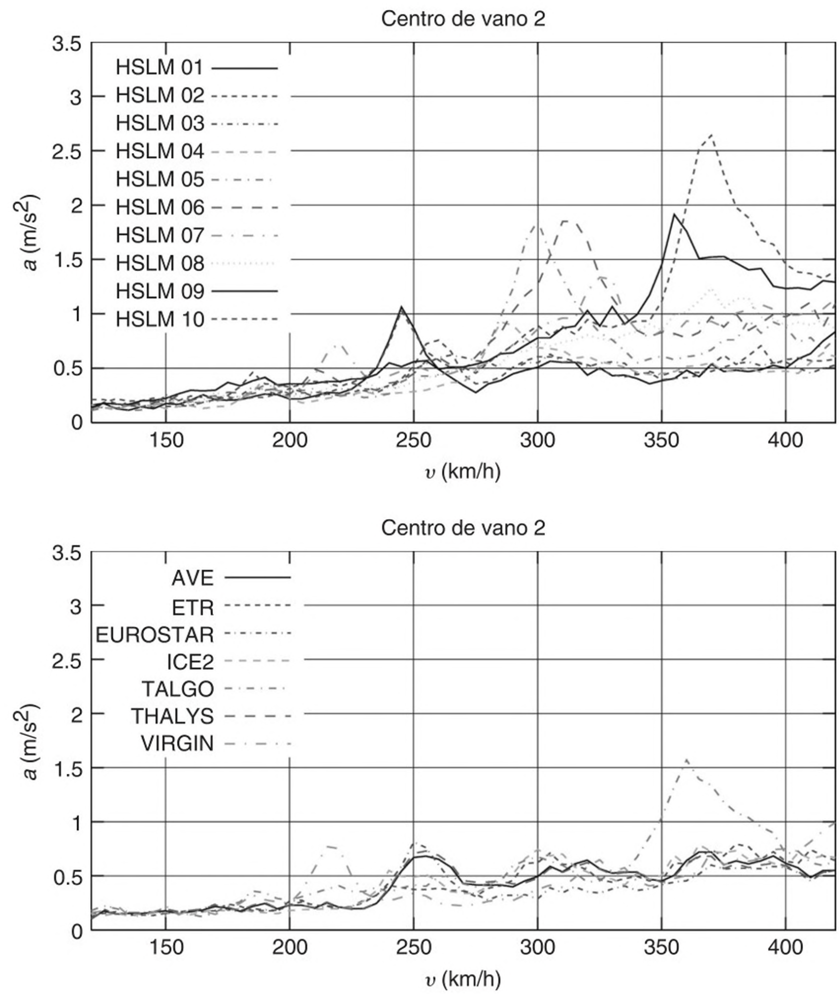

Figure 14. V2 viaduct. Maximum acceleration envelopes for universal trains HSLM-A and European real trains.

4.4 *Precasted concrete bridges*

About precasted concrete bridges, two typologies have been analysed: attached double box section bridges and double box section bridges (see Fig. 10).

The Río Moros viaduct, located in the Spanish Valladolid–Segovia high speed railway line, corresponds to the attached double box section typology. This viaduct has a total length of 475 m, with 11 continuous spans. The two side spans are 35 m and the intermediate spans are 45 m, having non–uniform side near the piers. Figure 11 shows a construction stage image and the finished viaduct.

The dynamic analyses were carried out using a finite element model with 3D beam elements based on the Bernoulli theory, coupling the bending and torsion effects. In accord to the Spanish IAPF code [14] seventy three modes of oscillation with eigenfrequencies lower than 30 Hz were considered for the computational analyses. A range of velocities from 120 km/h to 420 km/h every 5 km/h was considered for the analyses. The actions applied were the corresponding to the ten

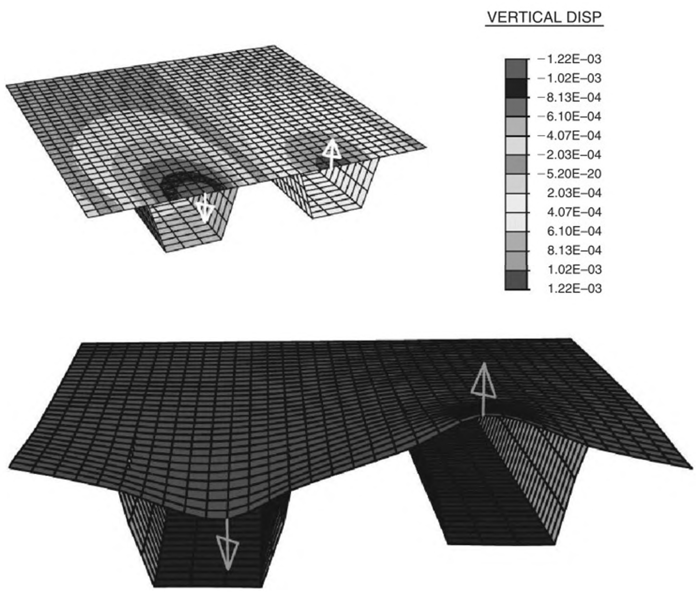

Figure 15. 3D FEM model of the Río Milanillos viaduct and deformed mesh with a magnification factor equal to 1000.

trains of the HSLM-A model (High Speed Load Model A) defined in [11,14], which are dynamic envelopes of the effects of the possible real trains. This load model assures the interoperability criteria, opening the possibility of interconnection with other European high speed lines.

For the double box section bridges two types can be distinguished corresponding to deck with and without longitudinal joint (see Fig. 12). Two precasted viaducts of this type have been analysed under high speed train loads, describing the main aspects of the results in the following paragraphs.

The V2 viaduct is a double box section bridge with longitudinal joint in its deck. It will be in the Spanish Madrid–Cuenca–Valencia High Speed line. At this moment this HS railway line is in project stage. This viaduct has three spans. The side spans are 16.5 m and the main span is 22.5 m. The calculations are carried out with one track loaded, taking into account the assessment of torsion effects associated to a minimum excentricity due to possible transversal displacements of the track [14]. The loads considered was the corresponding to the ten universal trains defined in the HSLM-A model [14,11], in the range of velocities from 120 km/h to 420 km/h. The results were compared with those obtained for the seven European real trains. In Figures 13 and 14 are showed the envelopes of impact coefficient (see expression (10)) and accelerations in the mid point of the main span, respectively.

It can be observed that, although designed to be dynamical envelopes for isostatic structures, the universal trains are envelopes for the impact coefficient Φ in this structure too. Nevertheless, it seems that the acceleration obtained for the Virgin train at 220 km/h is higher than those obtained for the universal ones. This aspect is not relevant because the values obtained with the Virgin and *HSLM*-07 train are similar, and because this intermediate peak is not determinant for design. Anyway, it is interesting to remark this fact because it is not proved the envelope capabilities of universal trains for statically indeterminate structures.

Within the group of bridges without longitudinal joint in the deck is the Río Milanillos viaduct. It is located in the Segovia–Garcillán HS line. Because its two box beams are separated 7 m and joined with a relatively flexible deck, the hypothesis corresponding to the infinity stiffness of the transversal section may be not correct for dynamical computations: the torsional eigenfrequency would be higher and in consequence the participation of this mode could be undervalued. In consequence, an equivalent torsional stiffness was assessed using to this end a 3D detailed finite element model. The FEM mesh has 1536 shell elements, resulting in a model with 10000 equations approximately. In order to impose a torsion moment two vertical point loads were defined in the axles of the tracks, and the warping of the supports and mid span was disabled. Figure 15 shows in the upper part the mesh with the contour of vertical displacements obtained and the point loads. In the lower part of this figure the deformed mesh is showed. Using an important magnification factor it can be shown that the hypothesis corresponding to the rigid solid movement of the transversal section is not suitable.

5 CONCLUSIONS

Nowadays the dynamic analysis of hyperestatic structures under high speed train loads is a requirement of the European design codes for railway bridges, because of the real possibility of resonance. To this end, several models based on direct integration with moving loads have been described in this paper. These models have been applied to some representative design problems, presenting a simplified methodology for the analysis of railway underpasses modelled as portal frames. Finally, some remarkable aspects corresponding to several dynamic analyses performed on real structures in the Spanish high speed railway lines have been discussed.

ACKNOWLEDGEMENTS

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CHAPTER 10

Bridge-vehicles dynamic interaction: numerical modelling and practical applications

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1 INTRODUCTION

Railway bridges, due to the high intensity moving loads to which they are subjected, are structures where the dynamic effects may reach significant values, which must be considered in the design. These effects are being given greater importance at present, in consequence of the increment on the circulation speed both in existing and new railways, as is the case for those intended for the high-speed trains. In high-speed railways, the dynamic effects tend to increase even more considerably, essentially as a result of the so-called resonance effects, which occur due to the passage of trains composed by several groups of regularly spaced axles.

The knowledge of these dynamic effects is of major importance for the case of railway bridges for the following reasons: i) the vibrations induced by the passage of the trains over the bridge originate, in general, displacements or internal efforts in the structures, greater than those produced when the loading is statically applied; ii) the excessive vibrations of the structure may lead to a magnification of the fatigue phenomena; iii) the deformations and accelerations of the bridge should be controlled and kept within certain limit values, in order to ensure the stability of the track and of the contact wheel-rail at all times; iv) the accelerations in the vehicles should be limited so that the passengers comfort can be guaranteed.

For a correct evaluation of the mentioned dynamic effects, it is necessary to have adequate analysis tools that enable to translate the complexity of the bridge-vehicle system in a realistic manner. In this work, a short description of the calculation program developed at the Faculty of Engineering of the University of Porto, where the bridge, the moving train and the respective interaction can be modelled, will be presented. Applications of this tool are described namely for the study of the dynamic behaviour of the Riada Bridge and the Antuã Bridge, conducted under the scope of the upgrading project of the Northern Line ("Linha do Norte") of the Portuguese Railways for the circulation of trains at higher speeds, as is the case of the CPA 4000 "Alfa Pendular" train, which can reach 220 km/h; of the Luiz I Bridge, which was carried out under the development of a technical study on the feasibility of its use for the passage of the New Light Metro of Porto; and of a continuous deck voided slab bridge, part of a high-speed railway line.

2 DYNAMIC MODEL OF THE BRIDGE-VEHICLES SYSTEM

The analysis of the dynamic response of bridges due to the passage of vehicles involves the consideration of the effects of a moving structure, the vehicle, on another immovable structure, the bridge. This study can be done, in a simplified manner, by simply considering a set of constant loads in movement or, using a more accurate approach, taking into account the interaction between the bridge and the vehicle. The first alternative can be justified in certain cases, but for a more exact translation of the phenomenon, it is necessary to involve the dynamic characteristics of the vehicle and the respective interaction between the two structures

This section describes the details of this last procedure, which was the basis for the development of a computer program for dynamic analysis of bridge-vehicle systems [1, 2, 3, 4].

The dynamic analysis of the bridge-vehicle system is performed during a certain period of time, by establishing separate dynamic equilibrium equations for the bridge and the vehicle, as follows:

$$\begin{bmatrix} M_b & 0 \\ 0 & M_t \end{bmatrix}\begin{bmatrix} \ddot{u}_b(t) \\ \ddot{u}_t(t) \end{bmatrix} + \begin{bmatrix} C_b & 0 \\ 0 & C_t \end{bmatrix}\begin{bmatrix} \dot{u}_b(t) \\ \dot{u}_t(t) \end{bmatrix} + \begin{bmatrix} K_b & 0 \\ 0 & K_t \end{bmatrix}\begin{bmatrix} u_b(t) \\ u_t(t) \end{bmatrix} = \begin{bmatrix} F_b(t) \\ F_t(t) \end{bmatrix} \tag{1}$$

where M, C and K are the mass, damping and stiffness matrices of the structure, F is the vector of external forces, u is the vector of nodal displacements and the indices "b" and "v" refer to the bridge and the vehicles, respectively.

The vertical action transmitted by the vehicles to the bridge F(t) contains a static component, F_{sta}, corresponding to the weight of the vehicles, and a dynamic component, F_{dyn} (t) , dependent on the interaction with the bridge:

$$F(t) = F_{sta} + F_{dyn}(t) \tag{2}$$

The moving loads course is constituted by a sequence of beam elements used for the structure discretisation. At the initial instant, it is necessary to define the position of the moving load in relation to the course origin. The position of the load at the instant t is obtained by adding the space run by the vehicle to the initial position. After defining the position of the vehicle the moving loads are converted into equivalent nodal forces and these nodal forces are added to the external load vector of the structure.

The differential equations of motion (1) are then solved using a numerical integration technique, like Newmark or Wilson-θ direct integration methods, and adopting an appropriate time step.

At each instant of time, the present formulation involves however an iterative procedure for the compatibility of the two structural systems, with the following steps:

i) The moving loads corresponding to the vehicle are applied on the bridge, the corresponding load vector $F_b^i(t)$ being obtained through the expression:

$$F_b^i(t) = F_{sta} + F_{dyn}^{i-1}(t) \tag{3}$$

where F_{sta} is the static component of the load force and $F_{dyn}^{i-1}(t)$ is the dynamic component of the interaction force at the last iteration (equal to $F_{dyn}(t - \Delta t)$ for the first iteration). The solution of the equations of motion of the bridge leads to values of the displacements at the nodal points $u_b^i(t)$and under the rolling loads;

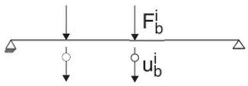
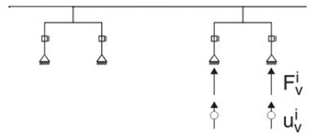
ii) At the same instant of time, the vehicles are submitted to the action of a support settings ($u_v^i(t)$) corresponding to the displacement under the moving load. The resolution of the equations of motion of the vehicle system permit to obtain the values of the corresponding support reaction ($F_v^i(t)$) which constitutes an interaction force $F_{dyn}^i(t)$ to be applied on the bridge at the next iteration;

iii) At the end of each iterative step, a convergence criterion is applied based on the values of the dynamic components of the interaction force at both the current and previous iteration. If the ratio

$$\frac{F_{dyn}^i(t) - F_{dyn}^{i-1}(t)}{F_{dyn}^{i-1}(t)} \tag{4}$$

is less than a given tolerance, the two structural systems can be considered compatibilized and the analysis proceeds to the next instant of time. Otherwise a new iteration is needed. This

Table 1. Methodology for the study of the interaction between the vehicle and the bridge.

	Bridge	Vehicle
Scheme		
Action	$F_b^i(t) = F_{sta} + F_{dyn}^{i-1}(t)$	$u_v^i(t) = u_b^{i-1}(t)$
Result	$u_v^i(t) = u_b^i(t)$	$F_{dyn}^i(t) = F_v^i(t)$
Convergence	$\frac{F_{dyn}^i(t) - F_{dyn}^{i-1}(t)}{F_{dyn}^{i-1}(t)}$	If $<$ tolerance $\rightarrow t + \Delta t$
criterion		If $>$ tolerance $\rightarrow i + 1$

(a) (b)

Figure 1. Riada Bridge – a) lateral and b) longitudinal views.

process begins by assuming that the dynamic component of the interaction force at the initial instant is equal to zero. Table 1 illustrates the described methodology.

3 RIADA BRIDGE

The entry and exit points of a railway bridge involve sudden changes of stiffness from the embankment to the bridge, forming critical points where very significant dynamic amplifications can occur. Such phenomena were investigated taking as case study the Riada bridge located at km 0 + 302.175 of the Linha do Norte of the Portuguese Railway Company (Fig. 1) [2, 5]. This bridge comprises two independent half decks, each of them supporting one of the railway tracks. Each half deck is formed by two steel beams, with a span of 6.60 m, supported at the abutment by steel bearings. The sleeper immediately before the bridge is placed directly on the masonry, an aspect that was taken in account in the numerical modelling.

Figure 2 shows the dynamic model used in the study. The model is composed by the following elements: i) continuous beam, of flexural stiffness $(EI)_r$ and mass per unit length m_r, simulating the rails; ii) springs, of stiffness k_{rs} and dash-pots of damping c_{rs}, simulating the pads located between the rail and the sleepers; iii) masses M_s simulating the sleepers; iv) springs, of stiffness k_{bsp} and dash-pots of damping c_{bsp}, simulating the ballast, sub-ballast and formation layers; v) springs, of stiffness k_{sb} and dash-pots of damping c_{sb}, simulating the pads located between the sleepers and the bridge; vi) beam, of flexural stiffness $(EI)_b$, mass per unit length m_b and structural damping factor ζ simulating the bridge.

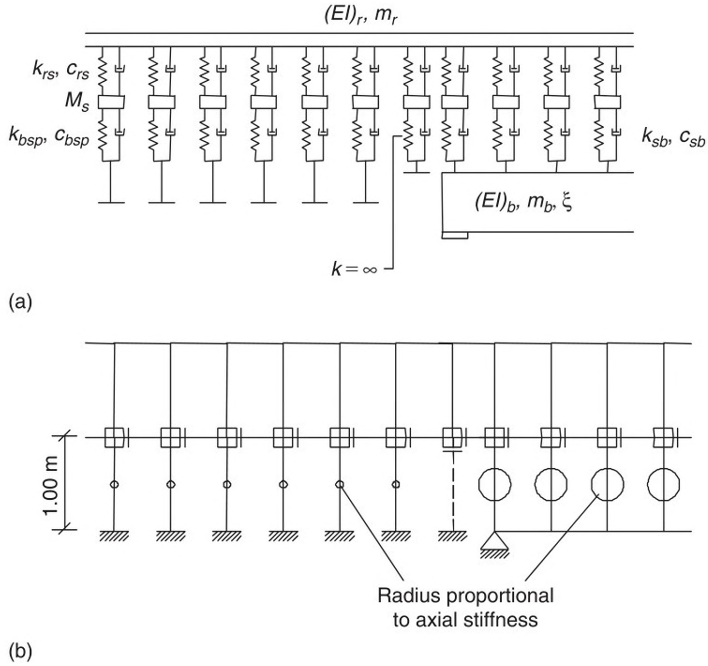


Figure 2. Embankment to bridge transition: a) dynamic model; b) discretisation using beam elements.

The train used in this study consists of an electrical BS5600 locomotive, in service in the CP network. The vehicle model adopted is the one used by Keymeulen & Winand [6] in work done for the ORE D160 Specialists Committee [7].

The model consists of the following elements: i) rigid body, of mass M_c, simulating the vehicle body; ii) springs, of stiffness K_s and dash-pots of damping C_s, simulating the secondary suspensions of the vehicle; iii) rigid bodies, of mass M_b, simulating the bogies; iv) springs, of stiffness K_p and dash-pots of damping C_p, simulating the primary suspensions of the vehicle; v) masses Me simulating the axles and the wheels and vi) springs, of stiffness k_h, simulating the elastic contact between the wheel and the rail (Fig. 3).

CP Rolling Stock Division supplied the values of the parameters of the vehicle model.

In Figure 4 the complete finite element mesh is showed. In total 39.60 m of the track were simulated, 26.40 m of which before the bridge, 6.60 m corresponding to the bridge and 6.60 after the bridge. At the beginning of the simulation, the first wheel of the locomotive BS5600 is at a distance of 9.60 m from the left end of the bridge; the simulation continues until the end of the finite element mesh is reached.

In addition to the sub-grade to bridge transition described (Situation 1), another two situations were analysed. In Situation 2, the stiffness of the element located under the four sleepers that precede the bridge is assigned a constant value which is an intermediate between the stiffness of the embankment, k_{bsp}, and the stiffness of the pad located between the sleeper and the bridge, k_{sb} (Fig. 2). In Situation 3, a linear variation between the two referred stiffness values is considered for the same four sleepers. In both situations 2 and 3, the support corresponding to the direct placement of the sleeper on the masonry abutment was eliminated.

For each of the transition situations referred to in the last section and for each of the considered train speeds, the dynamic component of the interaction force, F_{dyn}, between the first wheel of the locomotive and the rail was recorded. The records for the three analysed situations, for traffic

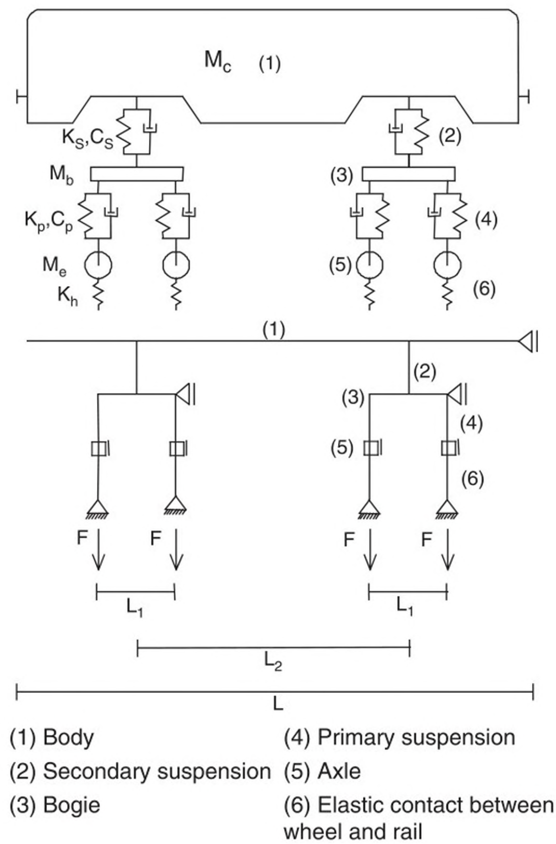


Figure 3. Dynamic model of BS5600 locomotive.

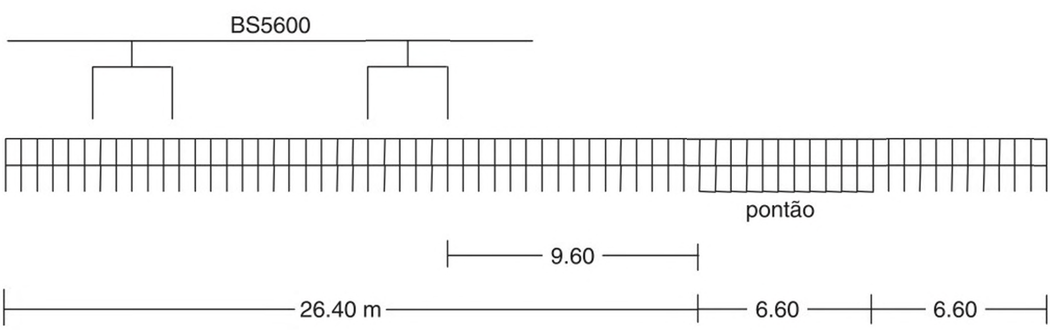


Figure 4. Finite element mesh.

speeds of 70.0 and 110.0 m/s are shown in Figure 5. The vertical dashed lines correspond to the location of the left and right supports of the bridge beams.

i) Situation 1
Through the analysis of Figure 5a, it can be seen that the maximum interaction force, indicated by the letter "M", occurs when the wheel is at a distance of approximately 1.0 m of the left bridge beam support. Subsequently the interaction force reaches a minimum, indicated by

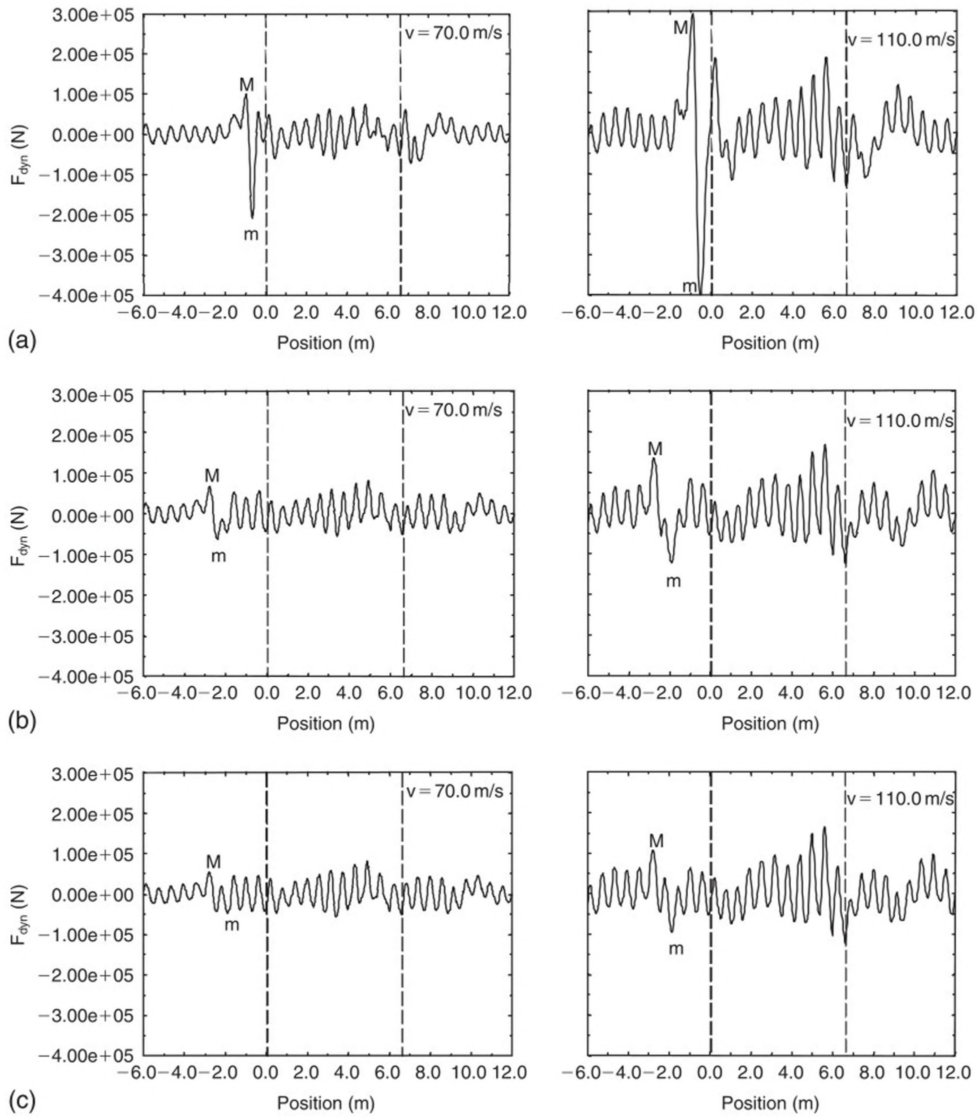


Figure 5. Dynamic component of the interaction force between first wheel and rail: a) Situation 1; b) Situation 2; c) Situation 3.

the letter "m", when the wheel passes on the point where the vertical stiffness of the track reaches its maximum value, i.e., 0.60 m of the left bridge beam support. The referred extremes increase with traffic speed. It was also verified that the maximum and minimum recorded in the transition from the bridge to the embankment were much inferior to those obtained in the transition from the embankment to the structure.

ii) Situation 2
The maximum and minimum of the interaction force recorded in the transition from the embankment to the structure occur now about 2.70 m from the left bridge support (Fig. 5b); this point seems to be associated to the vertical stiffness gradient of the track verified between the position −3.0 m and −2.4 m. However these extremes values are much lower to those recorded in the first situation.

iii) Situation 3
The maximum and minimum of the interaction force occur in a position identical to the one in Situation 2 (Fig. 5c); their values have the same magnitude of those recorded in that situation.

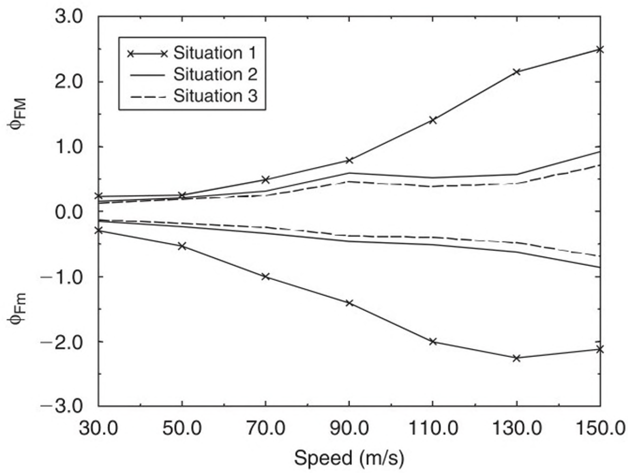


Figure 6. Maximum and minimum dynamic amplification of the interaction force.

In Figure 6, for each of the analysed situations and each of the considered train speeds, the following ratios are graphically represented:

$$\phi_{FM} = \frac{F_{dyn,M}}{F_{sta}} \tag{5}$$

$$\phi_{Fm} = \frac{F_{dyn,m}}{F_{sta}} \tag{6}$$

where $F_{dyn,M}$ and $F_{dyn,m}$ are respectively the maximum and minimum values of the dynamic component of the interaction force recorded during transition from the embankment to the structure.

These ratios represent the positive and negative dynamic amplifications of the interaction force between wheel and rail, which would be equal to F_{sta} in static conditions. Through the analysis of Figure 6, it can conclude that: i) the largest dynamic magnifications are always obtained in the first situation. The magnifications for the second situation are slightly superior to those obtained for the third situation; ii) There are not important differences between the maximum dynamic magnifications curves corresponding to the three situations up to speeds in the order of 90.0 m/s; iii) The gap between the minimum dynamic magnification curves for the first situation and for the others becomes more significant above 30.0 m/s (108 km/h); iv) For the first situation and the speed of 70.0 m/s (252 km/h), the minimum dynamic component of the interaction force cancels the static component ($\phi_{Fm} = -1$), which is a limit situation for the contact stability between the wheel and the rail. For traffic speeds above 70.0 m/s the existence of contact losses between wheel and rail ($\phi_{Fm} \leq -1$); v) was registered. For the second and third situations there was no contact loss between wheel and the rail ($\phi_{Fm} > -1$) up to the speed limit of 150.0 m/s (540 km/h).

4 LUIZ I BRIDGE

The Faculty of Engineering of the University of Porto has been responsible for the development of a technical study of Luiz I Bridge, a large XIX century metallic arch bridge for urban road traffic, in order to analyze the feasibility of its use for the passage of the New Light Metro of Porto [8, 9, 10].

This study involved, in a first approach, the consideration of only the static loads associated with different limit states. This exercise revealed the feasibility of using the upper deck of the

Figure 7. View of Luiz I bridge.

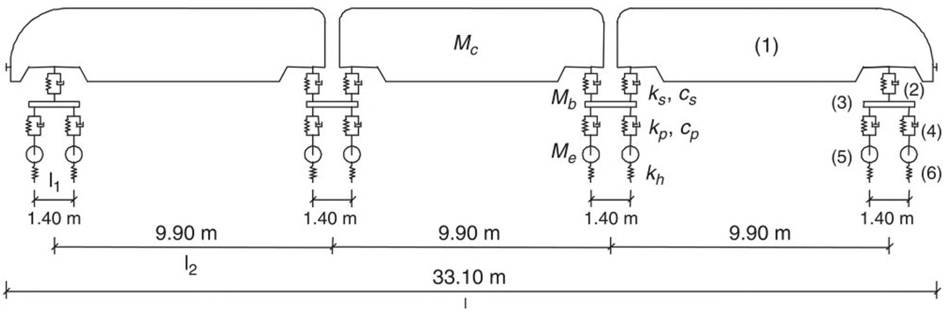


Figure 8. Dynamic model of the new light metro.

bridge to install a double line of the light metro, without any special traffic limitations, provided that some appropriate strengthening measures were undertaken. Subsequently, a dynamic analysis of the bridge was also performed, evaluating natural frequencies and mode shapes, estimating the dynamic response under seismic excitations and quantifying the dynamic effects due to the passage of the metro at different speeds, both in terms of structural safety and the comfort of pedestrians and passengers.

The dynamic model of the new light metro of Porto, used in the referred study, is schematically represented in Figure 8.

The discretisation of the bridge in beam elements is schematically represented in Figure 9, according to the 2D finite element model used. The idealization of the vehicles respected the dynamic model of the New Light Metro previously presented, a set of three vehicles having been considered together passing over the bridge.

The numerical simulations made considered five different values of the vehicles speed (30, 60, 90, 120 and 150 km/h), the structural response of the bridge being characterized in each time instant by the following control variables: vertical displacement and acceleration at the mid-span sections T1, T2, T3, T10, T11, T12 and T13, as well as of the arch, A, and vertical acceleration at the head (C1) and rear (C2) pivots of the first and last carriages of the Light Metro.

The results obtained from the numerical simulations of the dynamic response of Luiz I Bridge submitted to the moving loads of the New Light Metro were analyzed from two different points of view, namely: (i) in terms of structural safety, quantifying dynamic amplification factors associated

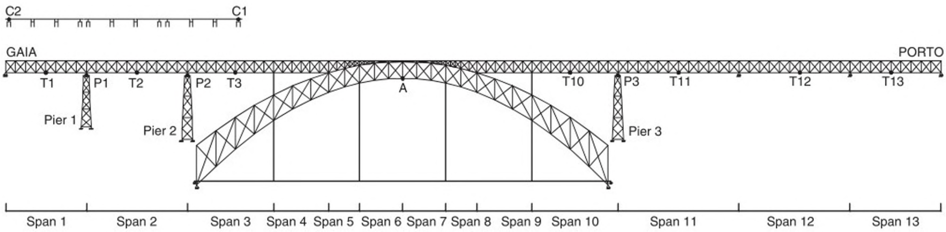


Figure 9. Finite element mesh with indication of the points of control.

with different structural elements (upper deck, arch and columns) and comparing them with values provided by the Eurocode 1 [11]; (ii) in terms of human comfort, both of pedestrians on the bridge and of passengers in the Metro, evaluating vertical accelerations at the upper deck and at the carriages, and taking into consideration appropriate maximum acceptable limits of vibration indicated by ORE [7] and OHBDC [12].

4.1 *Structural safety*

According to Eurocode 1, the dynamic effects induced by trains on bridges must be evaluated by multiplying the corresponding static effect, S_{sta}, by a suitable amplification factor, φ. The total effect is given by

$$S_{dyn} = (1+\varphi) \times S_{sta} \tag{7}$$

where $1+\varphi = 1+\varphi' + \varphi''$. In this expression, φ' is the component of the dynamic amplification factor associated with a perfect track, whereas φ'' represents the component directly related with track irregularities. The values of these two factors are given by the following equations:

$$\varphi' = \frac{k}{1-k+k^4} \tag{8}$$

$$\varphi'' = \frac{\alpha}{100} \cdot \left[56 \cdot e^{-\left(\frac{L_\Phi}{10}\right)^2} + 50 \cdot \left(\frac{L_\Phi \cdot n_0}{80} - 1\right) \cdot e^{-\left(\frac{L_\Phi}{20}\right)^2}\right] \tag{9}$$

where $k = v/2 \cdot L_\Phi \cdot n_0$, v is the train velocity (m/s), $\alpha = v/22 \leq 1$ (m/s), L_Φ is a reference length (m), depending on the deformability of the structural element considered, and n_0 is the fundamental frequency of the structure (Hz).

Following this procedure and neglecting track irregularities, it was possible to conclude that all the dynamic amplification factors obtained by the Eurocode 1 are higher than those directly evaluated by the numerical model developed for the purpose. This fact is evidenced by Figure 10, which shows a comparison between numerical and EC1 dynamic amplification factors, evaluated for the arch and for span 10, considering the 5 different vehicle speeds between 30 and 150 km/h.

4.2 *Pedestrian comfort*

The analysis of the comfort of pedestrians circulating on the upper deck was performed comparing the peak values of the vertical displacement at the mid-span of the several spans of the deck, as well as of the vertical accelerations with maximum acceptable values defined in the Canadian Code OHBDC [12].

Figure 11 presents a comparison of the peak values of vertical displacement for the several spans, at the recommended velocity of 60km/h, with the limits referred in that code, which are function of the fundamental frequency of vibration and of the degree of use of the bridge by pedestrians.

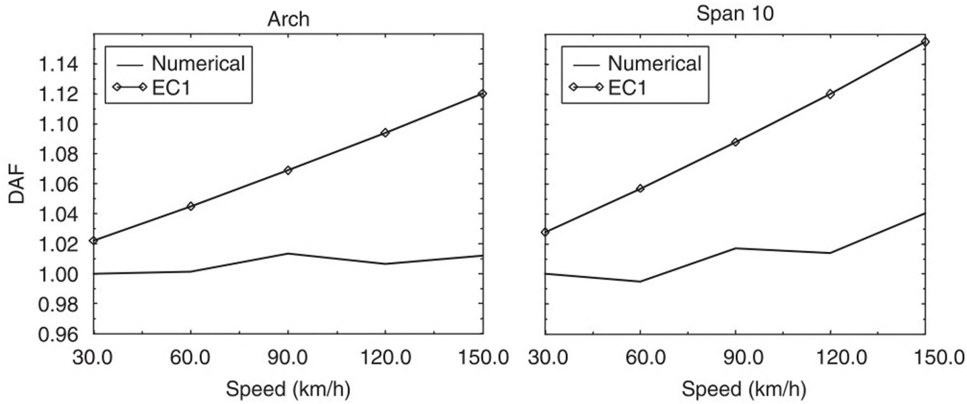


Figure 10. Comparison between numerical and Eurocode 1 [11] dynamic amplification factors.

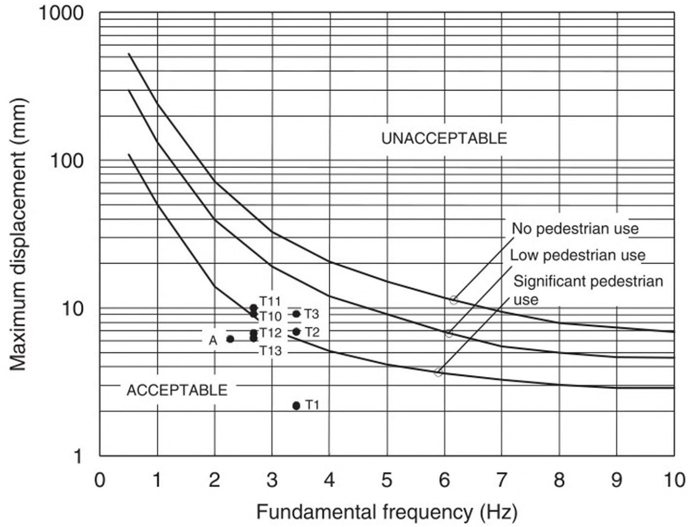


Figure 11. Comparison between maximum vertical displacements along the upper deck (v = 60 km/h) and the limits defined by OHBCD [12].

Inspection of this figure shows that, in case the bridge is not intensively used by pedestrians, there will be no problems of human comfort.

Figure 12, on the other hand, compares the peak values of vertical acceleration of the deck, for the vehicles speed of 60 and 90 km/h, with the limits defined by OHBDC, which are also dependent on the fundamental frequency of vibration. From careful inspection of this figure, it is also possible to conclude that, for the recommended velocity of 60km/h, the pedestrian comfort is acceptable, except for the first span. However, increasing the vehicle speed to 90 km/h, the level of human comfort is just acceptable for the arch and for spans 2, 10 and 12.

4.3 *Passengers comfort*

The analysis of the comfort of passengers of the New Light Metro was developed using the methodology suggested by the ORE experts' commission [7]. Following this procedure, the intensity level

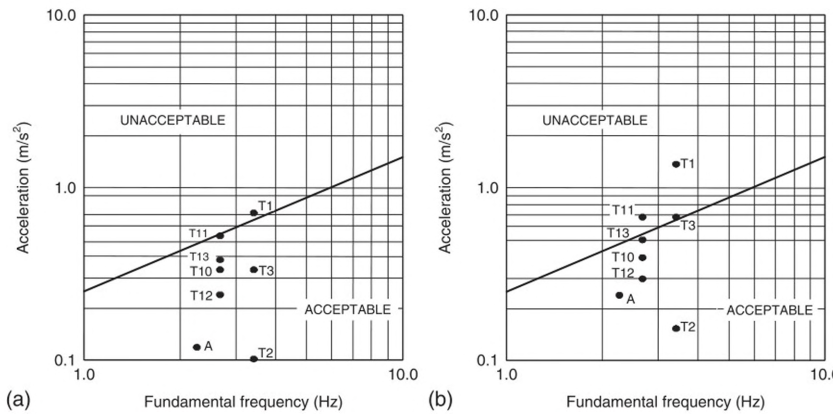


Figure 12. Comparison between maximum vertical accelerations along the upper deck and the limits defined by OHBCD [12]: (a) v = 60 km/h and (b) v = 90 km/h.

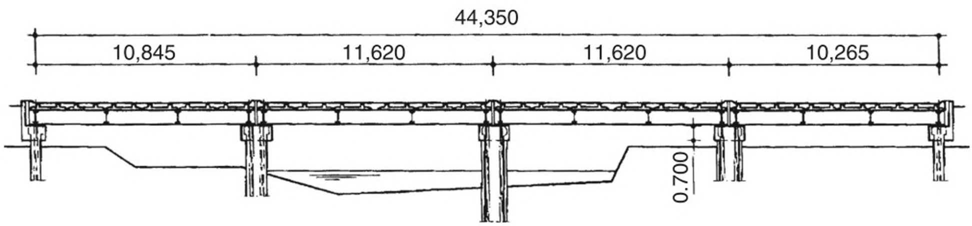


Figure 13. Longitudinal section of the bridge.

LI_h was evaluated using the records of vertical accelerations at the first and last carriages, for the velocity of 90 km/h, which led to $LI_h = 46.4$ in the former case, and to $LI_h = 53.7$, in the latter. Comparing these values with the corresponding maximum acceptable limits established by the ORE D160 commission ($LI_h = 95.0$ for $v \leq 120$ km/h and an excellent level of comfort), it was possible to conclude that the passengers comfort will be clearly excellent inside the New Light Metro.

5 ANTUÃ BRIDGE

The Northern Line "Linha do Norte" has been subjected to a number of interventions towards its renovation and to the possibility of circulation of the new high-speed trains, travelling at speeds greater than those for which the line had been initially designed. This is the case for the CPA 4000 train, which can reach 220 km/h.

The railway bridge over the Antuã River was subjected to one of those interventions, where the old deck, consisting of two independent half-decks comprising four spans, was replaced by eight half-decks in a concrete-steel cross-section structure, working as simply supported beams (Fig. 13).

In order to determine the dynamic effects associated with the crossing of the CPA 4000 "Alfa Pendular" train over the bridge, a study of its dynamic behaviour was undertaken [13, 14], which involved a set of dynamic analyses and the evaluation of the obtained responses in terms of structural safety, circulation safety and passengers comfort, according to guidelines established in EN 1991-2 [15] and EN1990-prAnnex 2 [16].

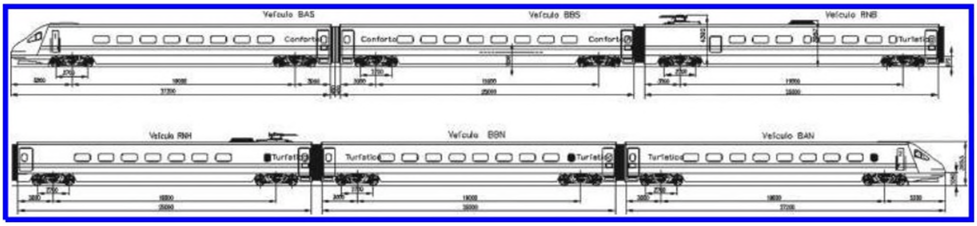


Figure 14. CPA 4000 train.

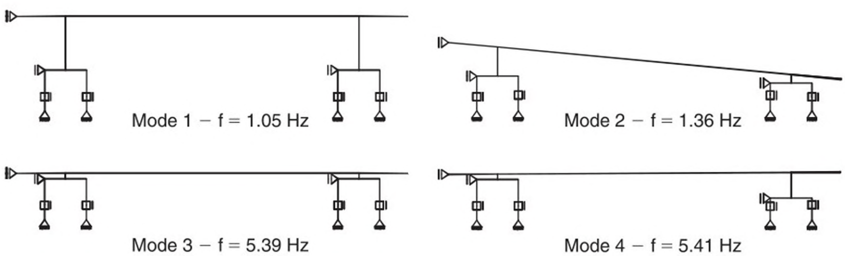


Figure 15. Schematic representation of the first four mode shapes of the vehicle.

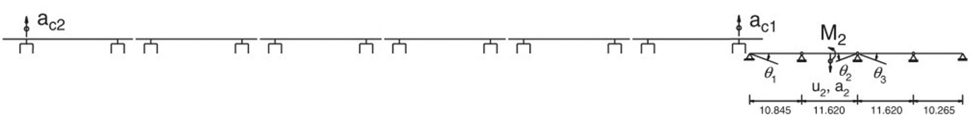


Figure 16. Finite element mesh with indication of the control parameters of the dynamic response.

The CPA 4000 train consists of a composition of six vehicles. Each vehicle has two bogies, each of them with two axles. The total weight of the composition equals 298.3 tonnes, tare weight, and 323.3 tonnes, in a normal loading situation. The maximum weight per axle is of 14.4 t. The total length of the composition is equal to 158.90 m (Fig. 14).

In Figure 15 are presented the frequencies and the configurations associated with the first four mode shapes of one of the vehicles. The first two mode shapes involve mainly the movements of the box (translation and rotation). The other two correspond to the movements of the bogies.

The dynamic analyses were conducted for speeds ranging from 140 to 300 km/h, at 10 km/h intervals. Such was considered in order to assess to the importance of this parameter, even for speeds greater than the maximum speed of the train, which is 220 km/h.

For each speed, time records were taken for all of the control parameters of the dynamic response indicated in Figure 16. These parameters are the displacement (u_2), the bending moment (M_2) and the acceleration (a_2) in the mid-span section of the 2nd span of the deck, the rotation angle (θ_1) at the endpoint of the 1st span of the deck, the relative rotation ($\theta_2 + \theta_3$) between the 2nd and 3rd spans of the deck and the accelerations (a_{c1} e a_{c2}) in the boxes of the first and last carriages.

Figure 17 presents the maximum values of the bending moment at the mid-span section of the 2nd span of the deck as a function of the speed. Careful inspection of this figure allows identifying the occurrence of peaks in the dynamic response for speeds values of 190 and 250 km/h.

The passage over the bridge of a train composed by several axles or groups of regularly spaced axles may, under certain conditions, induce resonance in the structure. Considering d as the regular spacing between the groups of axles, the speeds to which the resonance effects tend to occur are

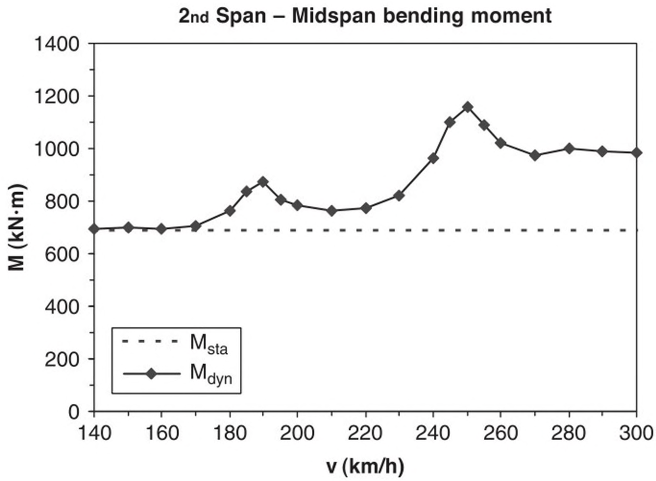


Figure 17. Maximum values of the bending moment at the mid-span section of the 2nd span of the deck as a function of the speed.

obtained by means of the following relation:

$$v_{res}(i,j) = \frac{dn_j}{i} \tag{10}$$

where n_j is the frequency associated with the jth mode shape of the structure and where i takes the values of 1, 2, 3, 4,... or 1/2, 1/3, 1/4... [17].

For the CPA 4000 train, the regular spacing between groups of axles is equal to 25.90 m. Hence, it is possible to verify that these velocities correspond to the excitation of the structure at frequencies equal to 1/3 and 1/4 of the frequency associated with the first mode shape of the structure ($n_1 = 8.16$ Hz), that is:

$$v_{res}(3,1) = \frac{25.9 \times 8.16}{3} = 70.4\,\mathrm{m/s}\ (\approx 250\,\mathrm{km/h});$$
$$v_{res}(4,1) = \frac{25.9 \times 8.16}{4} = 52.8\,\mathrm{m/s}\ (\approx 190\,\mathrm{km/h}).$$

5.1 *Structural safety*

According to EN1991-2 [15] the structural design should take into consideration the most unfavourable effects resulting from: i) a dynamic analysis of the bridge; ii) a static analysis of the bridge under the loading of the load model LM71 multiplied by the respective dynamic coefficient (Φ).

In what concerns i), the effects should be corrected in order to take into account the dynamic effects caused by the irregularities of the track. For that purpose, it is first necessary to determine the relative dynamic amplifications of each of the parameters relevant for the structural design, by means of the relation:

$$\varphi'_{dyn} = \frac{y_{dyn}}{y_{sta}} - 1 \tag{11}$$

in which y_{dyn} is the maximum value of the dynamic response and y_{sta} is the maximum value of the static response.

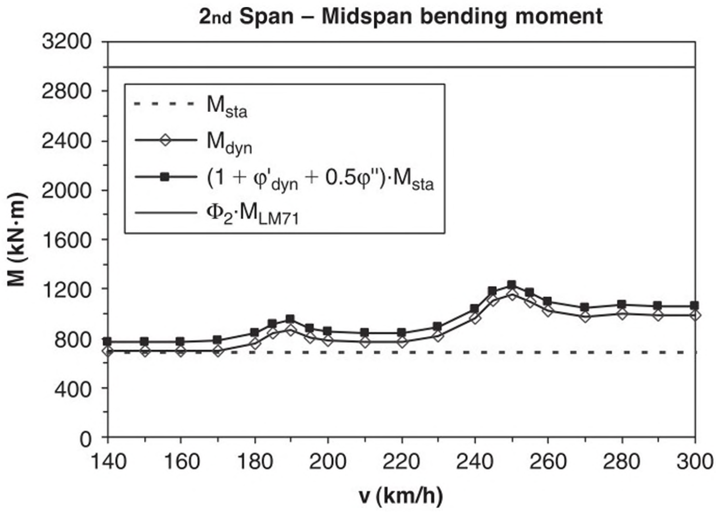


Figure 18. Comparison of the results of the bending moments at the mid-span section of the 2nd span of the deck, resulting from the dynamic analysis of the bridge and from the application of the load model LM71 multiplied by the respective dynamic coefficient.

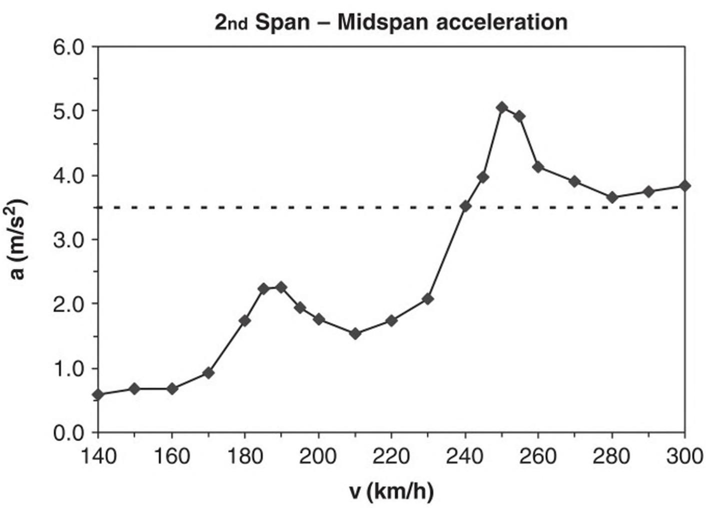


Figure 19. Maximum values of the acceleration at the mid-span section of the 2nd span of the deck as a function of the speed.

For the case of a railway where a particularly high level of maintenance can be guaranteed, the dynamic amplification obtained by relation 11 should be increased by $0.5\varphi''$. The effects to be considered in the design are subsequently obtained by means of the following relation:

$$(1 + \varphi'_{\text{dyn}} + 0.5\varphi'')y_{\text{sta}} \tag{12}$$

In Figure 18 are compared the results of the bending moments at the mid-span section of the 2nd span of the deck, resulting from calculations (i) and (ii). The figure shows that the maximum values of the bending moment obtained by means of the dynamic analysis are much lower that

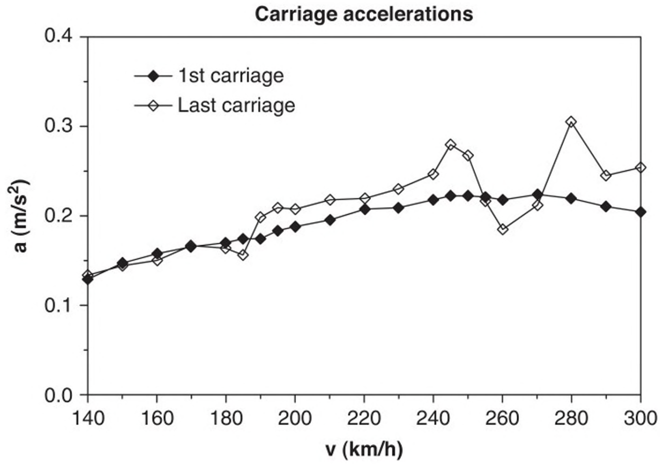


Figure 20. Peak values of the vertical acceleration in the interior of the boxes of the first and last carriages, as a function of the speed.

those resulting from the application of the load model LM71, multiplied by the respective dynamic coefficient.

5.2 *Traffic safety*

The EN1990-prAnnex 2 [16] establishes deformation and vibration limit states to be considered in the design of railway bridges. These limitations aim at ensuring the circulation safety and concern the: i) vertical acceleration of the deck; ii) torsion of the deck; iii) vertical deformation of the deck; iv) horizontal deformation of the deck.

In what concerns, for instance, the vertical acceleration of the deck, extreme values of this parameter may lead to the instability of the ballast or to the loss of contact between the wheel and the rail. The peak value of the vertical acceleration vertical for the case of a bridge with ballasted deck should not exceed $3.5\,\mathrm{m/s^2}$ ($\approx 0.35\,\mathrm{g}$).

In Figure 19 are presented the peak values of the acceleration obtained at the mid-span section of the 2nd span of the deck as a function of the speed. Analysing the graph, it can be observed that this value is exceed for speeds greater than 240 km/h.

5.3 *Passengers comfort*

The EN1990-prAnnex 2 [16] sets up limits for the peak value of the vertical acceleration in the interior of the carriages of $1.0\,\mathrm{m/s^2}$, $1.3\,\mathrm{m/s^2}$ and $2.0\,\mathrm{m/s^2}$ corresponding to the three levels of passengers comfort: Very Good, Good and Acceptable.

The peak values of the vertical acceleration in the interior of the carriages are shown in Figure 20, as a function of the speed. From the figure, it can be seen that the values are always lower than $1.0\,\mathrm{m/s^2}$ for any circulation speed, which correspond to a Very Good level of passengers comfort.

6 HIGH-SPEED RAILWAY CONTINUOUS SLAB BRIDGE

For speeds greater than 200 km/h, the dynamic effects tend to increase considerably, mainly due to the so-called resonance effects, which can strongly influence the structural solutions to be

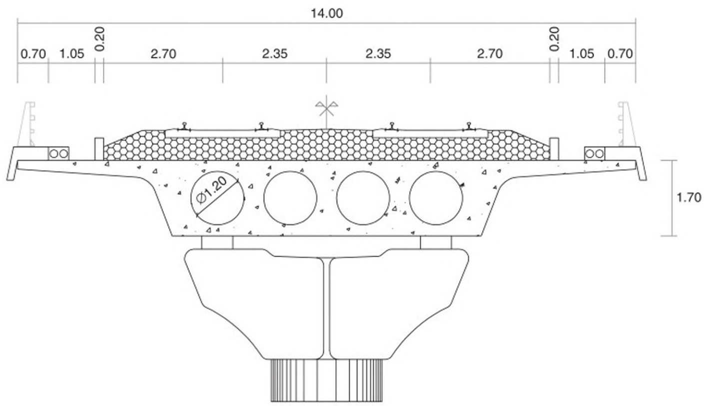


Figure 21. Cross section of the deck (voided slab zone).

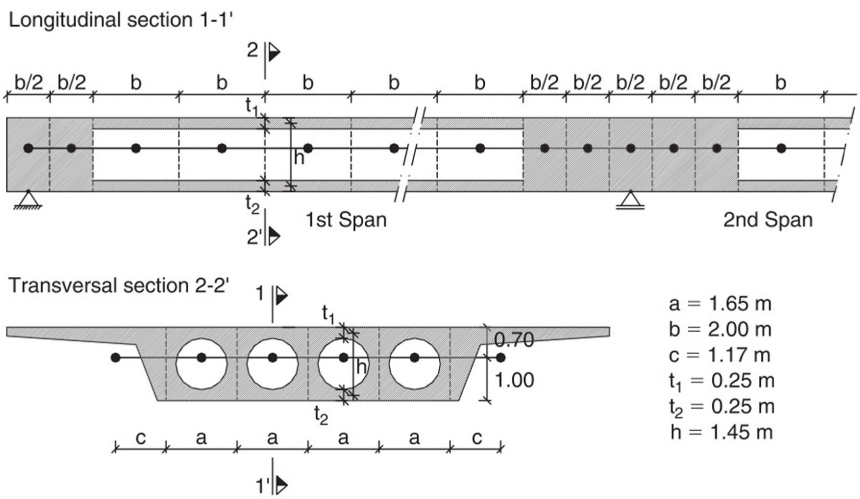


Figure 22. Longitudinal and transversal sections of the deck, with the location of the beam elements used in the discretisation.

adopted in the bridges. One of the solutions currently used for the cases of bridge spans around 20 and 30 m is that of a deck comprising a pre-stressed reinforced concrete slab, voided by means of cylindrical tubes [18]. The bridge in analysis consists of a continuous deck with seven spans (20.0 m + 5 × 25.0 m + 20.0 m) supported by support elements at the abutments and the intermediate piers. The deck is a pre-stressed reinforced concrete slab, working as support of both of the circulation railways. The slab is voided by means of four cylindrical tubes, of 1.20 m in diameter (Fig. 21), except near the piers and the abutments where it is solid.

The dynamic analyses were carried out based on a 3D model of the bridge, where the deck was discretized by beam elements forming a grid [19]. The location of the beam elements is indicated in the longitudinal and cross sections of the deck shown in Figure 22.

Analyses of the passage over the bridge of the ICE2 e EUROSTAR real trains were performed. The dynamic analyses were conducted for speeds between 140 km/h e 420 km/h (1.2 × 350 km/h).

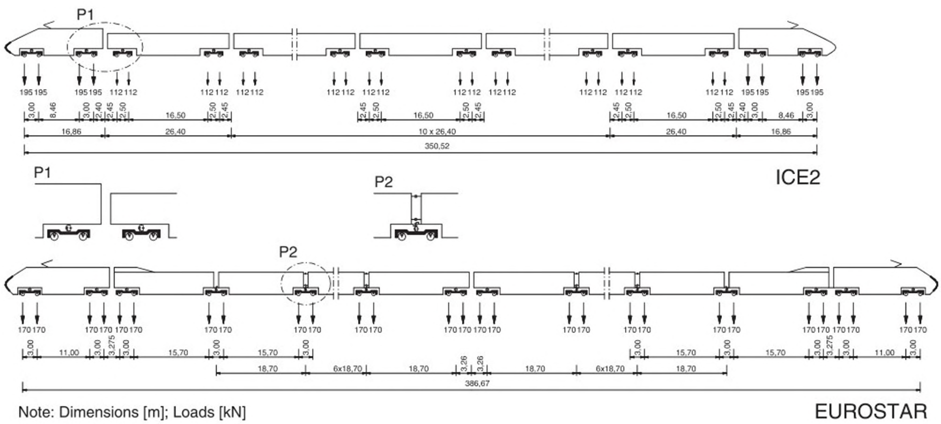


Figure 23. Dynamic models of the ICE2 e EUROSTAR trains.

The dynamic models adopted for the referred trains are presented in Figure 23. The parameters adopted for these models were obtained in [17].

The decision-making process about the need to perform a dynamic analysis of the bridge for the evaluation of the dynamic effects, including resonance effects, is presented in EN1991-2 [15] in the form of a flowchart (Fig. 24).

The decisions which lead, according to the flowchart presented in Figure 24, to the need of performing a dynamic analysis of the bridge were the following: i) the maximum speed at the bridge site was assumed equal to 350 km/h, hence greater than 200 km/h; ii) the bridge structure is not simple, since its behaviour is not analogous to a simply supported beam.

6.1 *Structural safety*

Based on the results of the dynamic analysis, the dynamic amplifications were determined through the following relation:

$$\varphi'_{\text{dyn}} = \max\left|\frac{y_{\text{dyn}}}{y_{\text{sta}}}\right| - 1 \tag{13}$$

where y_{dyn} is the maximum value of the dynamic response and y_{sta} is the maximum value of the static response obtained for the passage of each of the real trains RT.

In Figure 25 are compared the results of the bending moments at the mid-span sections of the 1st and 2nd span of the deck , obtained by means of the dynamic analysis of the bridge

$$(1 + \varphi'_{dyn} + 0.5\varphi'') \times \text{RT} \tag{14}$$

and those resulting from the application of the load model LM71 and SW/0, multiplied by the respective dynamic coefficient (Φ).

$$\Phi \times (\text{LM71}'' +'' \text{SW/0}) \tag{15}$$

From the observation of the figure, it can be concluded that: i) the bending moments obtained by means of the dynamic analysis of the bridge are much lower than those resulting from the application of the load models LM71 and SW/0 multiplied by the respective dynamic coefficient; ii) the highest bending moments were obtained for the load model SW/0.

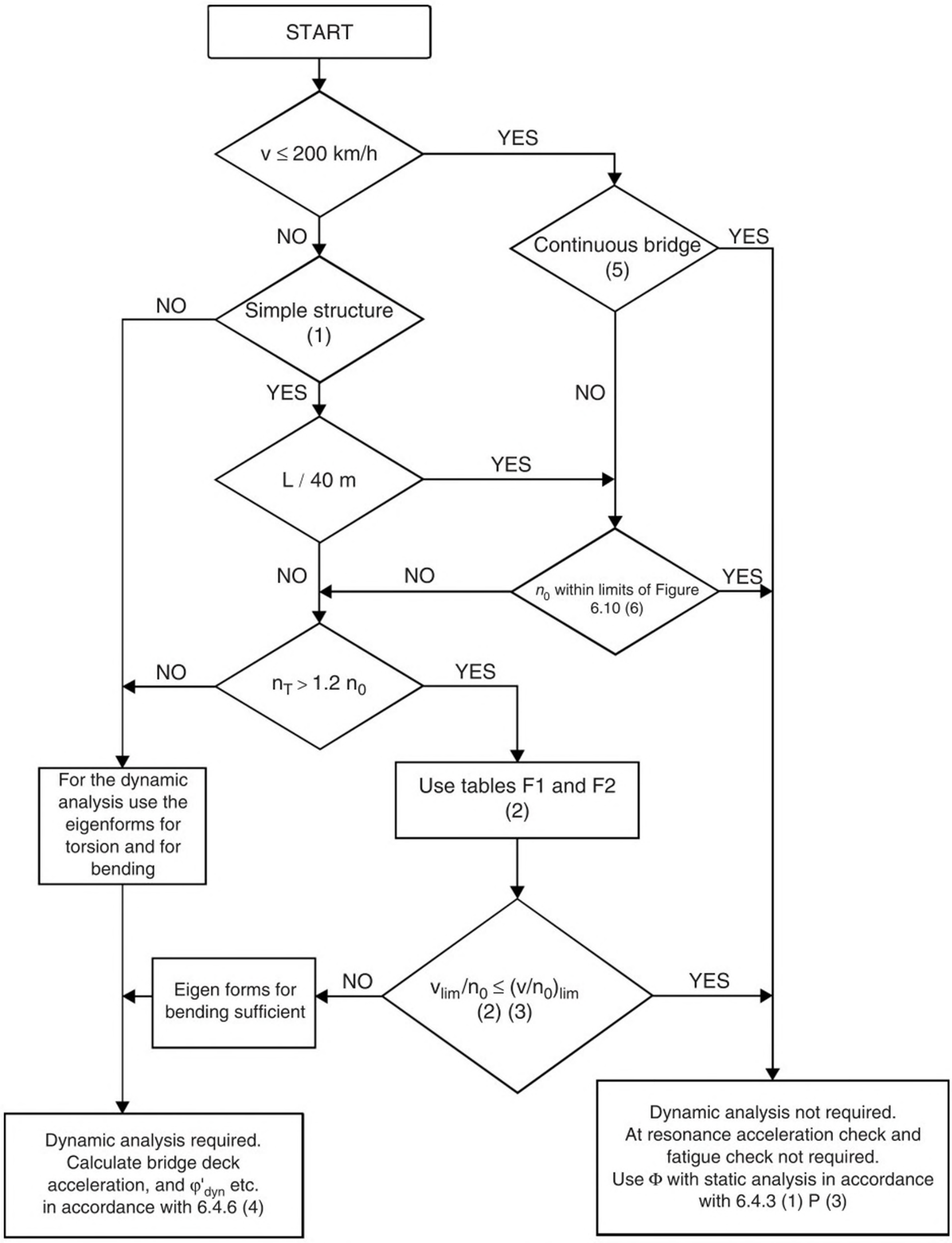


Figure 24. Flowchart for the determination of the need to perform a dynamic analysis of the bridge (adapted from [15]).

where:

v(km/h) is the maximum speed at the bridge site;
L(m) is the span of the bridge;
n_0 (Hz) is the fundamental bending frequency of the bridge;
n_t (Hz) is the fundamental torsional frequency of the bridge;
v_{lim}/n_0 (m) and $(v/n_0)_{lim}$ (m) are presented in the appendix F of EN1991-2.

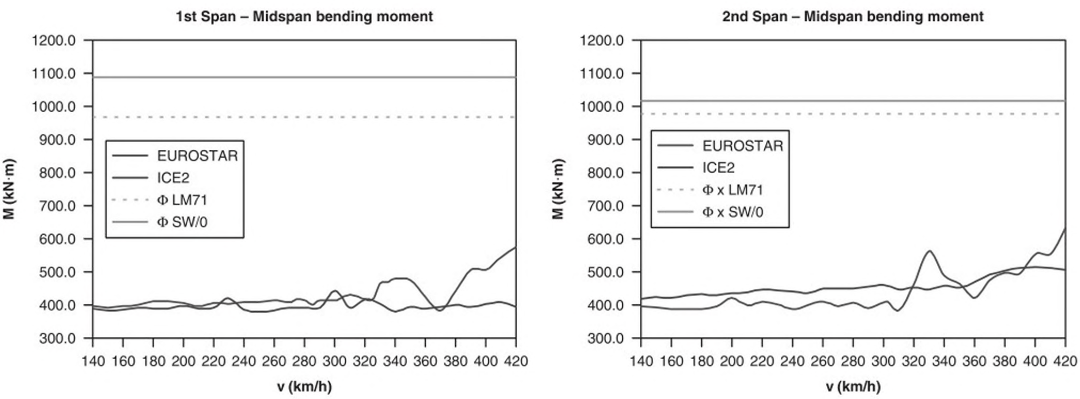


Figure 25. Comparison of the results of the bending moments at the mid-span sections of the 1st and 2nd span of the deck, resulting from the dynamic analysis of the bridge and the application of the load models LM71 and SW/0 multiplied by the respective dynamic coefficient.

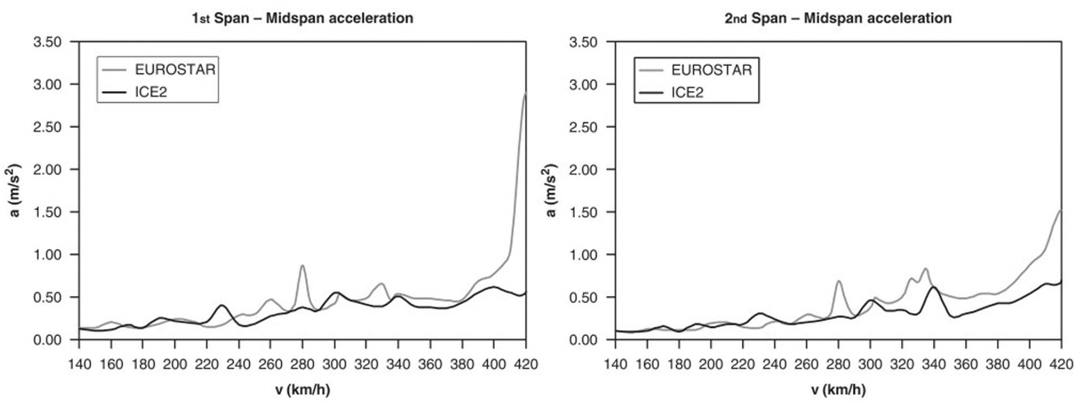


Figure 26. Maximum values of the acceleration in the mid-span sections of the 1st and 2nd spans of the deck, as a function of the speed.

6.2 *Traffic safety*

In Figure 26 are presented the peak acceleration values obtained by the mid-span sections of the 1st and 2nd spans of the deck, as a function of the speed. The analysis of the graphs reveals that the limit of 3.5 m/s^2 [16] has not been exceed in any of the analysed situations.

6.3 *Passengers comfort*

In Figure 27 are presented the peak values of the vertical acceleration in the interior of the carriages of the EUROSTAR train, as a function of the circulation speed. From the observation of the figure, it can be concluded that, according to [16], the passengers comfort was at Very Good level for all speeds.

7 CONCLUSIONS

Railway bridges are structures where the dynamic effects can reach significant values, which must be considered in the design. For a correct assessment of these dynamic effects, it is necessary to have the analysis tools that enable to translate in a realistic manner, the complexity of the bridge-vehicles

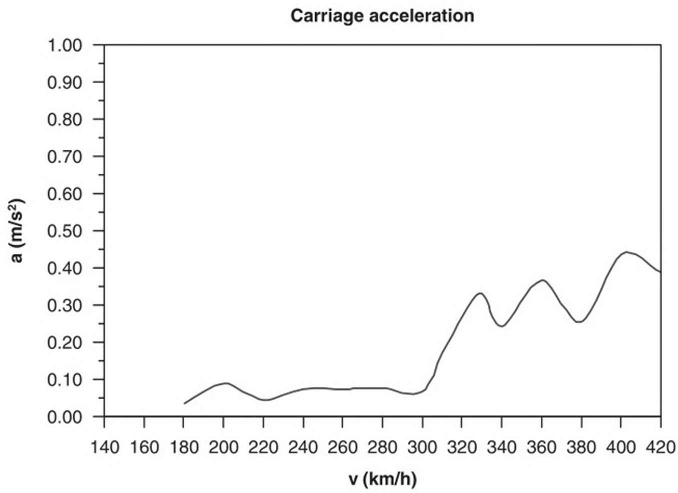


Figure 27. Maximum values of the vertical acceleration in the interior of the carriages of the EUROSTAR train, as a function of the speed.

system. An example of these tools is the numerical calculation program developed at the Faculty of Engineering of the University of Porto, where the bridge, the moving train and the respective interaction can be modelled. The relevance of this analysis tool was demonstrated by means of several applications to the dynamic behaviour study of bridges, namely for the Riada bridge and Antuã bridge studies, which were conducted under the scope of the upgrading and renewal works of the Northern Line ("Linha do Norte") of the Portuguese Railways for the circulation of trains at higher speeds, as is the case of the CPA 4000 "Alfa Pendular" train, which can reach 220 km/h; of the Luiz I Bridge, which was carried out under the development of a technical study on the feasibility of its use for the passage of the New Light Metro of Porto; and of a continuous deck voided slab bridge, part of a high-speed railway line.

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CHAPTER 11

Seismic design of structures in the French Mediterranean and Asian high speed railway lines

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We wish to mention that the chapter related to the monitoring of seismic activities for the Mediterranean TGV is based on Mr. Van-Tho Doan (Civil Engineer of the SNCF Bridge and Engineering Department) work and publications.

ABSTRACT: This paper deals with the seismic design of the High Speed Railway Line bridges, with a particular focus on the French Mediterranean and Asian High Speed Railway Lines which are both located in high seismic area. The first part of this paper presents the design philosophy for HSR Line. In particular, the two levels of earthquake to be taken into account: the high earthquake (design for repairable damage, where plastic hinges are allowed in the piers) and the medium earthquake (design for safe operation, where all elements shall remain in the elastic state and displacements are limited). In addition, the main seismic design issues are listed and commented. The second part of this paper focuses on the French Mediterranean and Asian High Speed Railway Lines. It details the seismic design philosophy, design criteria, seismic provisions, calculation methods, detailing construction requirement and monitoring.

1 INTRODUCTION

There is now a large experience of High Speed Railway bridge design in seismic area, thanks to the Japanese Shinkansen and the French Mediterranean TGV.

In seismic area with high magnitude, virtually all elements of the bridges are designed with the seismic loads. Nevertheless, in comparison to bridge design in non seismic area, the changes are not only in terms of quantities, but also in terms of design methods.

This paper focus particularly on two projects: the French TGV Mediterannean and Asian HSR.

2 PRESENTATION OF SEISMIC DESIGN FOR HSR LINE

During the design process of the French TGV Mediterranean, the assesment of the seismic risk lead to two main conclusions presented in the next sub-chapters:

- Railway bridges tend to be heavy, stiff and robust structures and are not well adapted to seismic conditions where light and flexible structures are preferred.
- A railway line extends on several hundred kilometers, which increases the seismic risk compared to an isolated building. In addition to the severe earthquake where structure failure and loss of life have to be avoided, a moderate earthquake with a short return period has been included in the

design in order to guarantee that no damage on the structures and the track can occur under this moderate earthquake and that under it the track geometry remains compatible with high speed train operation.

We also shortly presents in this chapter the main design issues for HSR bridge design in seismic area.

2.1 *Particularities of the railway line bridges*

The main characteristics of railway line bridges are:

- High permanent load due to the track and equipment loads.
  The track load varies from 170 kN/lm *for slab track to 250* kN/lm *for ballast track.*
- High bogie loads of the train and dynamic amplification factor.
- Fatigue effect.
- High braking and traction forces horizontal loads of the train.
  The fixed pier shall be able to resist such loads (up to 1600 kN *for a 30* m *long span according to the European practice).*
- Rail structure interaction
  The fixed pier shall be able to resist the longitudinal rail force if the track is cut at one end of the deck (around 550 kN/rail*).*
- High stiffness of the deck in flexion and torsion
  In order to satisfied the comfort criteria and maintain the safety requirements for railway bridges, the deformation of the deck are limited during train operation.
- High stiffness of piers
  In order to ensure the stability of the track, pier displacements are limited.
- Long life structure (100 years).

These requirements leads to very stiff and robust structures (for the decks, the piers and the foundations). This has an unfavourable effect for the seismic design where light and flexible structure are preferred to reduce the seismic load.

2.2 *Presentation of the medium and high earthquake level*

The primary purpose of earthquake design is to safeguard against major failure of the structure and loss of human life. This corresponds to the ultimate earthquake where the structure is allowed to respond into the inelastic range (within certain limit on the ductility ratio in order that all damage are reparaible). The ultimate capacity design is used for this severe earthquake loads.

In case of a High Speed Railway line, it is necessary to take into account in the design a moderate earthquake level (with a nominal ground acceleration level equal to around 1/3 of the severe earthquake level). Under this earthquake load, the structure shall be designed at the serviceability limit state and yielding of reinforcement and structural steel shall be avoided. The displacements and deformations of the deck and the track shall be checked to ensure that the train operation can be restored in a short time after the earthquake.

Moderate earthquake are expected to happen several time during the operation of the line because their return period is short (around 50 years). In addition, the return period of an earthquake corresponds to the seismic risk on one isolated structure. Due to the long length of the line (several hundred of kilometers), the return period of moderate earthquake for the total line can be significantly shorter.

Therefore, long and expensive structure and track repairs (which include the loss of revenue due to the traffic interruption) cannot be allowed under moderate earthquake.

It is to be kept in mind that ductility ratio for the piers can reach very high values (3.0 was used in the longitudinal direction for the Vernegues viaduct in France). Therefore, the moderate earthquake with serviceability design (track geometry) may be more critical in terms of quantities than the severe earthquake with capacity design.

2.3 *Main seismic design issues*

In this chapter, we list the main seismic design issues that we faced during the bridges design in Asia and in France. We shortly comment each item.

2.3.1 *Soil liquefaction*

Soil liquefaction may occur in some sandy soil. There are two main possibilities for the design:

– Deep foundation anchored well below the soil subjected to liquefaction
– Treatment of the soil subjected to liquefaction.

Both solutions can be used. Nevertheless, the soil treatment is more adapted for light structures and low earthquake loads.

If there is a risk of soil liquefaction, this has a very important impact on the design. Therefore, the main difficulty is probably to reach an agreement on the liquefaction risk between the engineers in charge of the design and the checking. Numerous methods exist to assess the liquefaction risk (French AFPS, Seed method, Ishihara and Yoshimine method, . . .) and may lead to different results. To avoid long and costly discussions between experts, it is recommended to define clearly which method shall be used by the designers to assess the soil liquefaction risk.

2.3.2 *Pier*

The weight of the HSR bridges and their pier stiffness have an unfavourable impact on the earthquake loads. They are significantly higher than for similar highway bridges.

Damping devices can be used in order to reduce the loads on the piers. Such systems are expensive, but very efficient for long continuous structure. It is to be noted that the system works if there is a relative displacement between the pier (or abutment) and the deck. Therefore, the pier shall be sufficiently stiff in order to limit their displacement under earthquake. On the French TGV Mediterranean line, the damping devices were placed on the abutments. For this reason, they may not be adapted for a succession of short simple spans (in addition to the high cost of placing damping devices on each structure).

For continuous deck, it is preferrable to have under normal operation only one fixed bearing, in order to avoid stresses in the deck and loads on the fixed piers due to the temperature variation. In this case, the fixed pier resists to the earthquake loads of the full structure. Lock Up Device can be used between the pier and the deck, so that under slow motion, the relative displacements between the pier and the deck are possible, and under fast motion, the pier can be considered as a fixed pier. The earthquake loads can be then distributed on all the piers equipped with Lock Up Device.

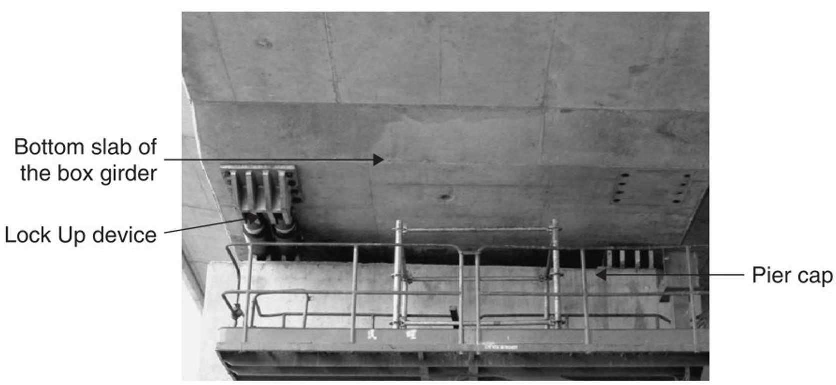

Figure 1. Lock Up Device located between the deck and the pier cap – Front view.

Figure 2. Lock Up Device located between the deck and the pier cap – View from below.

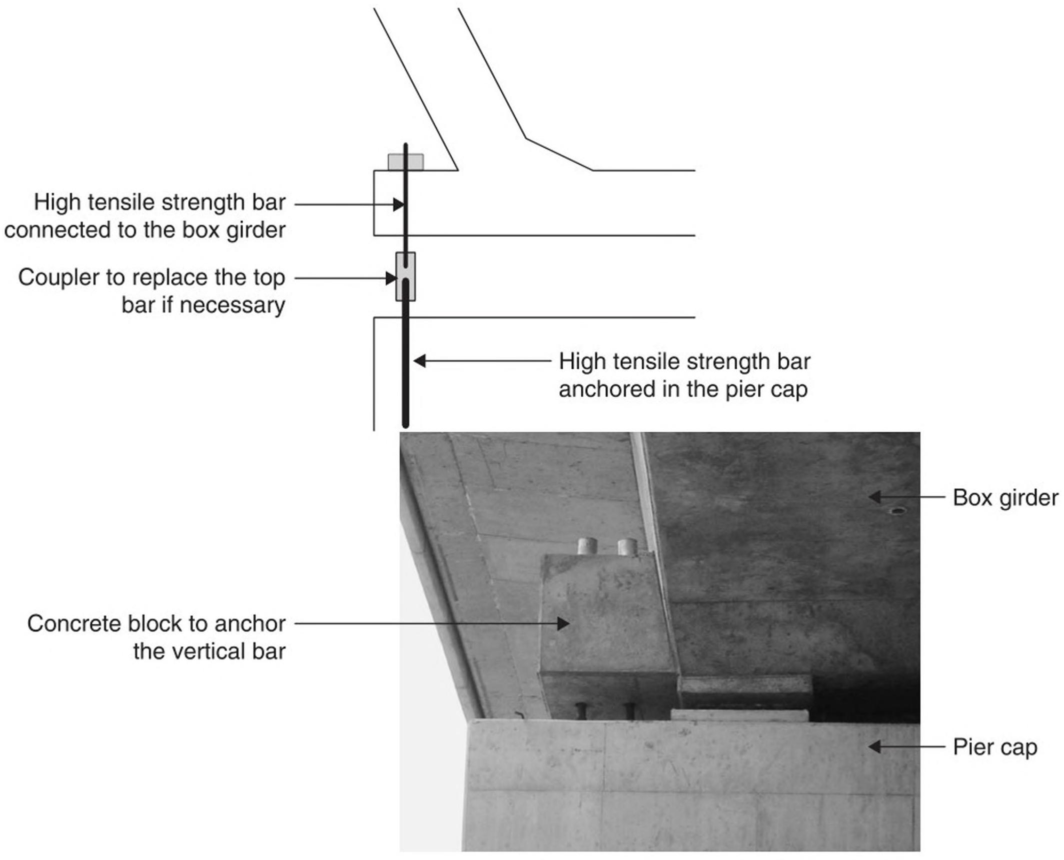

Figure 3. The hold down device located next to the bearings prevents the overturning of the deck.

In terms of design procedure, the difficulty is to reach the best compromise between the necessary stiffness of the pier to satisfy the commercial operation requirements and the flexibility of the pier to have the lowest earthquake loads.

2.3.3 *Deck*

In high seismic area, the reinforcement in the deck depends directly on the earthquake loads. This leads to very high reinforcement ratios in the structure (up to 230 kg/m^3) and difficulties to pour the concrete are common, specially in the diaphragm of prestressed box girder structures. It is

Figure 4. The hold down device located next to the bearings prevents the overturning of the deck.

necessary to ensure on the detailed design drawings that the reinforcement necessary can be placed and correct concrete pouring is possible.

In case of high transverse earthquake loads, the stability of the deck may not be ensured. Hold down devices can be placed in order to guarantee that no overturning of the deck is possible. Vertical high tensile strength rebars connect the bottom slab of the deck and the pier cap. These bars shall not prevent the rotation of the bearings and the relative displacement between the deck and the pier.

In case of a concrete box girder, the torque inertia of the girder may be much higher than the flexural inertia of the piers. In this case, even if the decks are disconnected at each structural joint, a continuity of the torque moment may be assumed from one deck to the other throught the pier cap. If there is a large variation of the pier flexural stiffness, this may leads to a concentration of torque moment in the decks near the stiffer piers. It is therefore recommended to avoid large variation of pier stiffness or to guarantee that fissuration of the concrete happened (which reduces the torque inertia). Three dimensional multimode FEM analysis is required for a proper design.

2.3.4 *Standardization of the design solution*

Various design interpretation and solutions are available for the bridges in seismic area: from the interpretation of the soil parameter (stiffness, bearing capacity, liquefaction, . . .) to the damping or lock up devices.

It is recommended that the calculation methodology and mechanical systems allowed or forbidden to be clearly defined and standardized before the designers start the design of the bridges.

Having large discrepancies in the design solution will complicate the supervision of the design process. Higher maintenance cost can also be foreseen if various mechanical devices are used.

2.3.5 *Track structure interaction*

The longitudinal displacements of bridges under normal operation are limited in order to satisfied the track safety requirements. This leads to stiff piers which is unfavourable for the earthquake design. Detailed track structure interaction analysis allows the designer to optimize the pier design for normal operation and thus, to reduce the pier stiffness. Such approach needs long calculations which are difficult to coordinate with the seismic analysis calculation (in both cases, the piers cannot be studied seperately, but as a part of the long viaduct, the computer model are therefore very big and calculations are long to be carried out). Nevertheless, the optimisation process can lead to significant cost savings.

Under moderate earthquake, the track stability is to be ensured. No calculation procedure is widely accepted to perform the track structure interaction analysis for the moderate earthquake loads. Two approachs have been seen:

– Limit the relative displacement between two structures under moderate earthquake. In this case, the calculation does not take into account the track structure interaction in order to simplify the calculation and avoid time history analysis.

– Perform complex time history analysis which includes the track structure interaction and the non linear behavior of the track.

The second method shall be handled carefully. Indeed, by modeling the track and the bridge in the time history analysis, a link between two adjacent decks is created through the track and the rails. This link reduces significantly the relative displacements between the two decks and have a favourable impact on the seismic behavior of the viaduct. Nevertheless, the results depend largely on the mechanical characteristics of the track which are not guaranteed in case of earthquake (and not necessary well defined at this stage of construction).

In addition, the purpose of the track is not to participate to the structural resistance of the bridge, but to support safely the running train. Therefore, this approach shall be considered carefully and it is to be checked that it does not lead to very favourable results. The safety of the bridge shall not rely too much on the track resistance.

3 SEISMIC DESIGN ON THE FRENCH HSR LINE TGV MEDITERRANEAN

3.1 *Presentation of the project*

The French HSR Mediterranean line is located in the south of France. The length of the line is around 250 km. More than 200 km of the line is located in seismic area. There are 390 bridges on the line (85 road bridges and 305 railway bridge), including more than 20 long bridges. The maximum span length is around 125 m and some bridges have 60 m heigh piers.

The design speed of the line is 350 km/h.

3.2 *Design criteria*

The French regulation requires to perform a capacity design of the structure with different nominal accelerations of the ground depending on the location of the bridge:

Zone Ia $a_N = 1.5\,m/s^2$
Zone Ib $a_N = 2\,m/s^2$
Zone II $a_N = 3\,m/s^2$

In addition to the French regulation requirements, the French national railway compagny SNCF checked the effects of a lower magnitude earthquake ($a_N = 0.65\,m/s^2$) on the structures at the Serviceability Limit State. The criteria were the following:

– Maximum stress in the structure: inferior to f_e (f_e elasticity limit) for the steel and inferior to $0.75\ f_{cj}$ (f_{cj} compressive strength) for the concrete.
  The structure stays in the elastic range under a moderate earthquake. No yielding of the concrete and the steel is allowed.
– Maximum relative longitudinal displacements between two adjacent decks or one deck and one abutment of 20 mm (when there is no rail expansion joint) under longitudinal earthquake to garantee the safety of the track.
  The track structure interaction is not included in the calculations.
– Maximum relative transverse displacements between two adjacent decks or one deck and one abutment of 20 mm under transverse earthquake. The angular variation between two decks or one deck and one abutment shall not be higher than 0.003 rd and the horizontal curvature radius shall not be lower than 9500 m.
– Under vertical earthquake, the maximum vertical acceleration shall not be higher than $7\,m/s^2$.

3.3 *Method of analysis*

For regular and standard bridges, the monomodal analysis was carried out. For more complex bridges, a multimodal analysis with a response spectrum was carried out. For the long bridges with damping devices which have non linear mechanical behavior, time history analysis were performed based on accelerograms of the seismic area.

In the transversal direction, when the irregularity of the bridge were important and it was not possible to ensure that all the piers can yield at the same time, a ductility ratio equal to 1 was applied. In the longitudinal direction, ductility ratio up to 3.0 were applied.

3.4 *Construction seismic specifications*

The connection between the deck and the pier is made by a steel or concrete shear key which works in longitudinal and transverse direction. The purpose of these connection is to ensure the integrity of the bridge and they shall satisfied the following requirements:

- Fiability: the transmission of forces shall be as simple as possible
- Easy to built: the type of shear keys depends on the construction method of the deck and of the material used to build the pier and the deck
- Possibility to inspect, repair and replace, if necessary.

Stringent requirements for the reinforcement are defined: minimum percentage of reinforcement, minimum spacing, higher minimum overlap length, ...) in order to ensure the confinement of the concrete in the critical part of the pier.

The damping devices require large space to be placed on the abutment. Their length can be up to 3 m and the amplitude of the displacement can reach 600 mm.

The structural joint between two adjacent structures shall be sufficient large to prevent shock between structures under earthquake. No damage on the structural joint shall happen under moderate earthquake. Therefore, adjustment in the longitudinal and transverse direction shall be possible.

3.5 *Monitoring of the seismic activities*

The purpose of the monitoring of the seismic activities is to limit, or even stop the traffic in an automatic way when a quake occurs. It is also used to assess the earthquake level and decide if basic or detailed inspections are necessary.

This monitoring network is based on the structure of the telecommunications network of the SNCF and includes:

- 24 Local Stations of measurement distributed along the line
- 1 Central Station Located in Marseille
- 1 Station of Confirmation located at the CEA (French Atomic Energy Authority)
- 2 telecommunications systems linking the various Stations.

The local monitoring ensured at each line section is independent; it is connected to a centralised system of decision-making, to form the seismic inspection network.

The sensor is made up of an accelerometer with 3 components (vertical, longitudinal and transverse) whose only vertical component is used for alarms, two others being used for the adjustment of the threshold of measurement.

In order to meet the conditions imposed by the RAMS of the Mediterranean HSL, the network is entirely redundant, from the connections of the measurements acquisition box, until the earthquake monitoring centre (EMC) located in Marseille. These two branches of the architecture are identical and completely independent, as well as the operation and maintenance desks and the station of confirmation of the CEA.

For each sub-system (local station or EMC), the average time between two consecutive failures is higher than 4500 hours. There should not be more than one minor false alarm (0.4 m/s^2) every 20 years, nor more one major false alarm (0.65 m/s^2) every 30 years.

The 24 Local Stations are spaced every 10 kilometres and distributed along the Mediterranean HSL. They measure the acceleration level (accelerometer according to 3 axes) in the vicinity of the line to compare it with thresholds of detection.

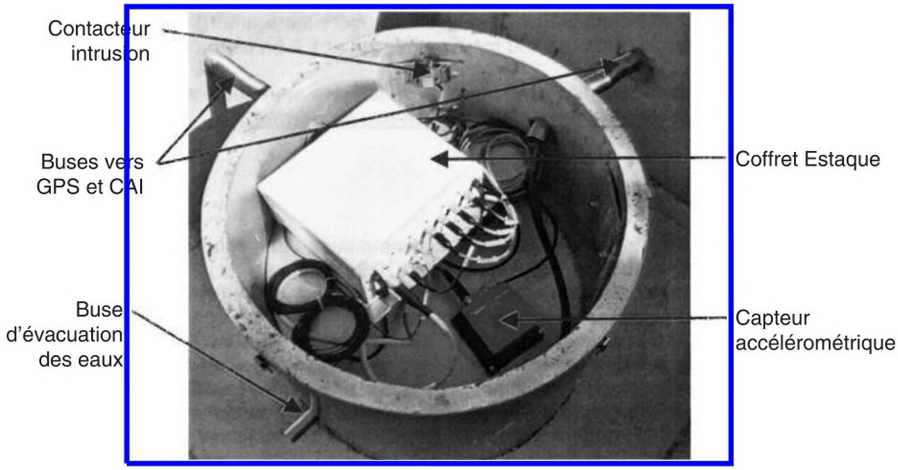

Figure 5. Local station: tub containing sensors.

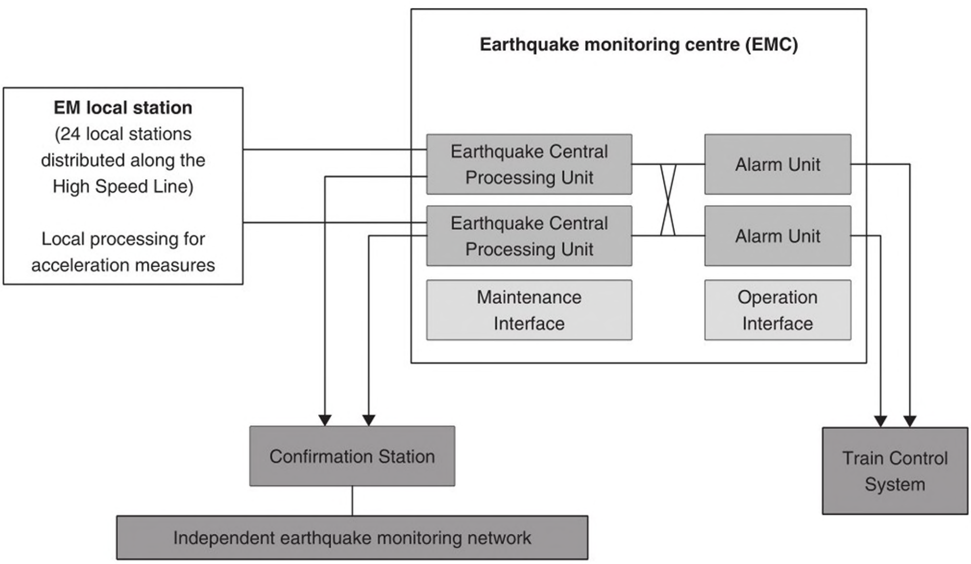

Figure 6. Architecture overview.

According to the result of this comparison, a message of detection and the data are transmitted by the Local Station towards the EMC established in Marseille via the two redundant telecommunications networks.

The level of the pre-alarm thresholds is adjusted according to the attenuation laws and of the effects of site. According to the result, messages of beginning and end of pre-alarm are transmitted, by the Local Station, towards the EMC.

The EMC which includes 2 processing units records the data and events of maintenance and has the responsibility of generating alarms.

Each of the two processing units of the EMC translates and checks the received messages, checks the level of detection compared to the thresholds and carries out the space and temporal correlation of pre-alarm and detection messages according to a logic of detection.

The declaration of the various alarm levels is done then according to the logic of following decision:

– if 3 contiguous stations in operation sent a corresponding pre-alarm message in a time interval of 5 seconds, the corresponding alarm (major or minor) is activated,
– a station out of service is considered on the same level as its neighbours,
– an alarm is broadcast for each triplet of stations having exceeded a given threshold.

The EMC is in charge of sending a message to the dispatcher to slow down with 170 km/h (minor alarm 0.4 m/s^2) or to stop (major alarm 0.65 m/s^2) the trains. It emits simultaneously a request for confirmation of the seism towards the Seismological Centre of the CEA.

The restart conditions of the circulation of the trains and the measures to be taken are given according to the response of confirmation or not-confirmation coming from the Seismological Centre which has its own seismic network (14 stations). This one must be emitted in the 10 minutes following detection.

The station of confirmation has the role, on request of the EMC, following release of a minor or major alarm, to analyse the seismic signals of CEA stations (independent of the Seismic Inspection Network) established in the south-eastern quarter of France to confirm or not the existence of a seism which can affect the zone concerned with the alarm.

An automatic software of detection and localisation of the earthquake allows, starting from the received message of the EMC, including the time of release of alarm, to locate and determine the importance of the event responsible for the alarm or to invalidate the alarm.

4 SEISMIC DESIGN ON HSR LINE IN ASIA

4.1 *Presentation of the project*

HSR lines in Asia are planned and are currently under construction. They include long portions of bridges (up to several hundred kilometers) and high ground nominal acceleration (up to 0.40 g).

4.2 *Design criteria*

Two types of earthquake have been defined: the "repairable damage" earthquake (severe earthquake) and the "safe operation" earthquake (moderate earthquake). Major failure and loss of life shall be prevent under the severe earthquake and capacity design is performed.

The nominal acceleration of the moderate earthquake is equal to 1/3 of the one of the severe earthquake. Under moderate earthquake, no yielding can happen and displacements of the structures are limited. The displacements requirements are as follow:

– Maximum relative longitudinal displacements between two adjacent decks or one deck and one abutment of 25 mm (including the displacement of one train braking).
– Maximum vertical and angular angular change under the moderate earthquake and the train load (including dynamic impact) on one track are limited to the value given in Table 1.

4.3 *Method of analysis*

Three types of analysis were allowable:

– Equivalent static analysis method for regular and standard bridges
– Dynamic multi-modal analysis method
– Time history analysis method.

Under severe earthquake, the track structure interaction is ignored. But, for moderate earthquake with the time history analysis method, the track structure interaction can be taken into account. Nevertheless, the track characteristics were not defined with accuracy at the time of the design.

Table 1.

Span length (m)	Vertical angle (10^{-3} rad)	Horizontal angle (10^{-3} rad)
10	1.7	1.7
20	1.7	1.7
30	1.5	1.7
40	1.3	1.3
50	1.3	1.3

Considering that the the time history analysis with track structure interaction results depend largely on the track mechanical characteristics, such approach was not widely used.

4.4 *Construction specifications*

The construction specifications are generally based on the AASHTO and local design code, which are similar than the one used for the French TGV Mediterranean (minimum reinforcement ratio, confinement reinforcement, . . .).

5 CONCLUSION

Bridge design in seismic areas raises several design issues (soil liquefaction, shear key design, displacement requirements under moderate earthquake, . . .) that have been studied for the previous High Speed Railway project in France and Asia. They do not present major technical difficulties. Nevertheless, the seismic design philosophy and experiences varies from one country to the other and, on international project, the difficulty to reach a agreement on the design methods between the experts shall not be underestimated. The design process can be significantly slowed down and the technical compromises reached may not be completely satisfactory.

CHAPTER 12

Closed and open joints for bridges on high speed lines

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ABSTRACT: Bridges, and especially the ends of bridges with closed and open joints, disturb essentially railway tracks. This area has a bigger importance in high speed tracks. Special geometric constructions can avoid these problems.

Keywords: *Closed and open joints, displacements and rotations at ends of bridges, slab track; ballasted tracks, anchors; upper and lower joint level, bearings, compensation plate.*

1 INTRODUCTION

Bridges, and especially the ends of bridges with closed and open joints, cause considerable disturbances to the railway. In the case of high speed lines fitted with slab track, the bridge ends are of special importance because the displacements and rotations as well as possible high forces in the rail fastenings that may develop in this area can be problematic. The geometric tolerances and tension and compression forces have a significant influence on the design of closed and open bridge joints for slab track.

For ballasted tracks there are few problems at the bridge ends as the ballast tolerates the movements of bridge structure due to temperature as well as creep and shrinkage by rearrangement of the ballast stones.

As the length of high speed lines constructed with slab track increases, more attention needs to be given to the problems at the bridge ends, and to the joints in particular, where only limited tolerance of rotations and displacements can be permitted.

1.1 *Definitions*

The area of the joint is divided into two levels. Because the vertical and horizontal forces are transmitted from the rail to the structure, a distinction in the definition of the joint levels has to be made in the case of slab track and ballasted track.

In the upper joint level, there is a gap in the permanent way. In this level, provision can be made for a bridging construction (such as a compensation plate described in point 4.2).

In the lower joint level the bridge and supporting structure are separated. At this level provision is made for the drainage system.

2 MOVEMENTS AT THE JOINT

At the end of the superstructure there are movements due to deflections of the superstructure. The longitudinal and lateral displacements can be quite substantial with the result that the rail fastenings are subject to high forces.

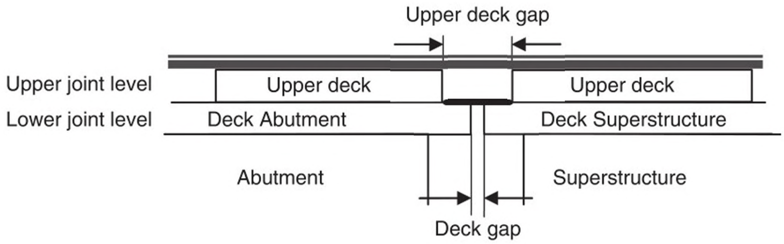

Figure 1. Definition of upper joint level and lower joint level.

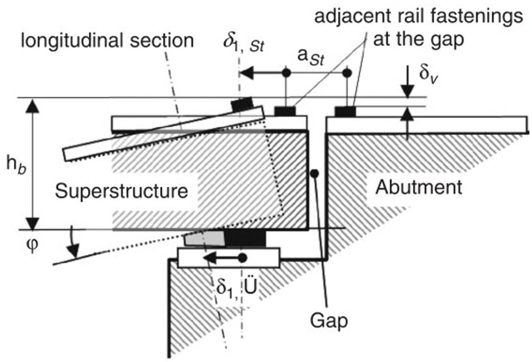

Figure 2. Longitudinal movements.

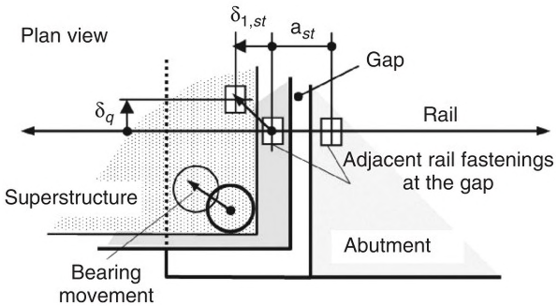

Figure 3. Lateral movements.

φ; Angle of rotation at the end of the superstructure

The rotation is due to the vertical movement of the superstructure caused by its own weight, imposed load, creep and shrinkage, and vertical temperature differences.

$\delta_{l,St}$: Longitudinal movements of the rail fastenings

The longitudinal displacement of the rail fastenings is a result of the linear contraction of the superstructure caused by temperature effects, creep and shrinkage (Dilatation $\delta_{l,\ddot{U}}$) and the angle of rotation φ.

$\delta_{l,St} = \delta_{l,\ddot{U}} + \varphi \cdot h_b$

$\delta_{l,\ddot{U}}$ => only at joints with moveable bearings.

δ_q: Lateral movements at the end of the superstructure

With lateral moveable bearings, horizontal displacements of the superstructure occur due to centrifugal forces, wind forces and horizontal temperature differences.

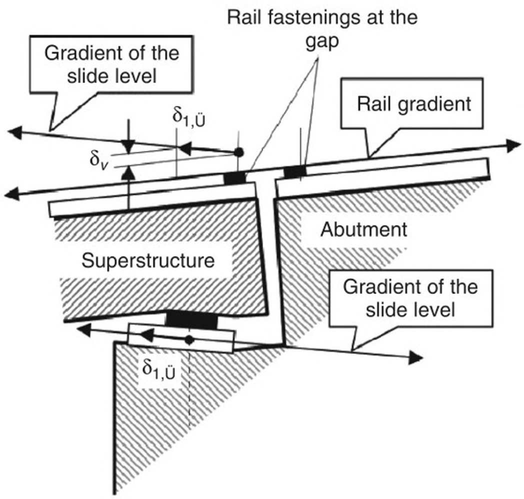

Figure 4. Vertical displacement due to the gradient.

δ_v: Vertical displacements at the end of the superstructure

The vertical displacement is a result of the angle of rotation φ, which causes a vertical movement of the rail fastenings at the end of the superstructure.

Furthermore the vertical displacement can be caused by a longitudinal movement $\delta_{l,Ü}$ when the sliding surface of the bearing is not parallel to the rail gradient.

A difference in the gradients could also be due to inaccurate installation of the bearing.

Example: In the case of a longitudinal movement $\delta_{l,Ü} = 20$ cm, a vertical displacement of $\delta_v = 1$ mm can result due to an installation tolerance of only 0.5%.

Additionally a vertical displacement of the superstucture can be caused by an inclined support under the bearings. This may be the result of differential settlements of the pier.

2.1 *Possible movements at the joint*

Bridgelength	Possible movement $\pm\Delta$ a	Open or closed joint
20 m	+25/−6	closed
100 m	+113/−30	closed or open
200 m	+219/−60	open
400 m	+406/−120	open
600 m	+565/−180	open

The table gives an immagination about movements of bridges. The movements results from a calculation and are very realistic.
+ means a reduction.
− means an elongation.

2.2 *Influence of displacements on the rail*

Additional stresses in the rails and in the rail fastenings of continuous welded rails are caused by movements at the end of the superstructure as shown in the figures.

Stresses at the rail fastenings adjacent to the joint

The angle of rotation φ, and especially the vertical displacement δ_v between the adjacent rail fastenings at the joint, result in large compression and tension forces.

A small vertical displacement of $\delta_v = 1$ mm, generates tension-forces of 10 to 17 kN at the rail fastenings (it depends on distance of the rail fastenings and the bending stiffness of the rail).

The stresses in the rail fastenings, caused by displacements at the end of the superstructure, must not exceed the allowable stresses. If the stresses are exceeded, a constructive measure (for example

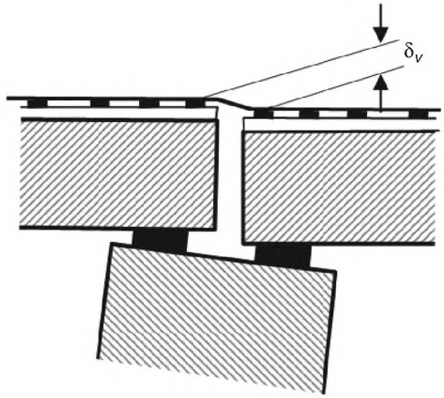

Figure 5. Vertical displacement due to inclined pier.

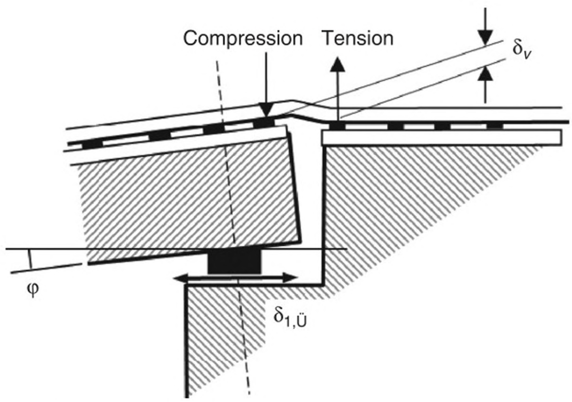

Figure 6. Forces at the rail fastenings near the joint.

compensation plates) is needed to ensure the serviceability of the rail. The lateral allowance δ_q is only 1 mm (see Anforderungskatalog FF, Deutsche Bahn). This is the reason for the high precision requirements in bearing tolerances and installation.

2.3 *Stresses in the rail near the joint*

Horizontal movements $\delta_{l,St}$ at the end of the superstructure cause additional rail stresses. The additional allowable rail stress must not be exceeded at the joint. If that cannot be achieved the bridge needs an adjustment switch to reduce the rail stress. Concrete bridges do not normally need an adjustment switch for superstructures less than 90 m.

2.4 *Distances between the rail fastenings at the moveable joint*

The distance between the rail fastenings at the moveable joint changes permanently, caused by longitudinal movements $\delta_{l,St}$ of the superstructure, especially in the case of prestressed concrete bridges, due to creep and shrinkage. The distance between rail fastenings should not exceed 650 mm. If the distances get bigger a bridging construction for the upper joint level is necessary.

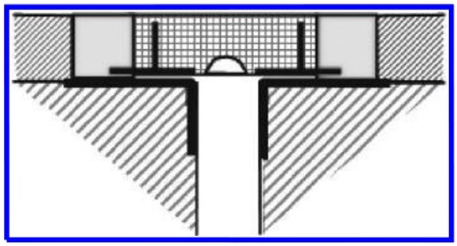

Figure 7. Closed joint at the lower joint level.

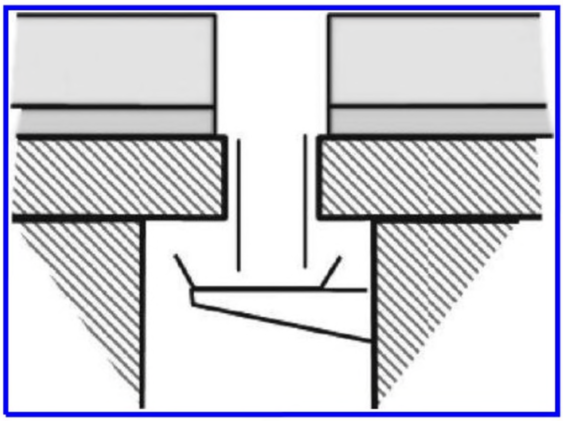

Figure 8. Simple drainage construction.

3 CLOSED JOINTS

Closed joints are normally used at the lower joint level. If there is a closed joint no bridging constructions are necessary at the upper joint level since no longitudinal movements are caused by temperature differences and creep and shrinkage of the superstructure. Closed joints are used at abutments with fixed longitudinal bearings, at piers with two fixed longitudinal bearings and at moveable joints with a length of superstructure up to 90 m.

A closed joint at the gap

Closed joints at fixed longitudinal bearings or moveable joints with an opening range of ±65 mm.

The advantage of using closed joints is the small installation width. This facilitates a small distance between the upper deck and the rail fastenings. To achieve easy maintenance, the front of the upper deck should be bevelled.

4 OPEN JOINTS

An open joint occurs when the end of the superstructure can move longitudinally at the support for example at abutments and piers. Temperature differences and creep and shrinkage cause longitudinal movements and angles of rotation.

Open joints should be separated in the upper and in the lower joint level, because high rail forces may occur at the ends of the superstructure. Calculations of rail stresses are necessary so as to decide whether a bridging construction is required at the upper joint level.

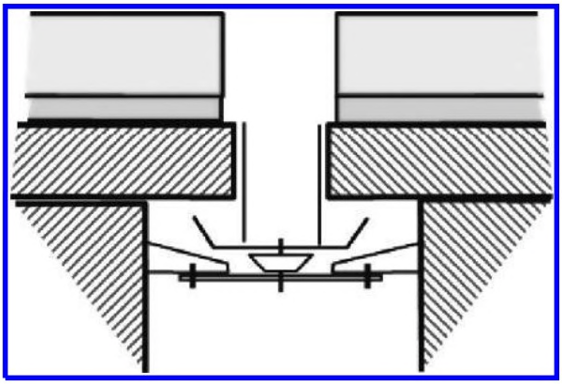

Figure 9. Kinematic drainage construction.

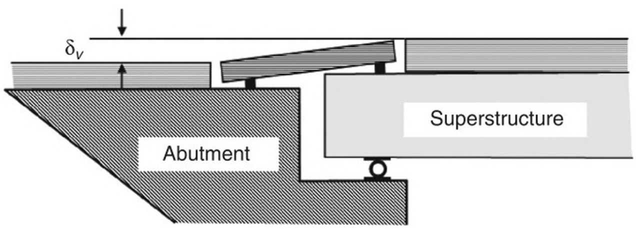

Figure 10. Compensation Plate – Longitudinal section.

4.1 *Open joints at the lower joint level*

There are normally two types of open joints. The function of the lower joint level is to drain the deck of the bridge.

Open joint at the lower level for bridge lengths of up to 300 m.

This construction is a simple drain construction. With drain-mats it is guaranteed that the water will fall into the drain.

Open joint at the lower level for bridge lengths exceeding 300 m.

This construction is a kinematic drain construction which centres the drain in the joint.

4.2 *Open joints at the upper joint level (compensation plates)*

A compensation plate is a small bridge in a composite construction that crosses the joint at the end of the superstructure at the upper joint level. The compensation plate is fixed in the lateral direction and can move longitudinally. It reduces the negative effects of the vertical displacement δ_V at the joint and eliminates the lifting forces in the rail fastenings near the joint.

Additionally, the distance between the rail fastenings on either side of the joint can be reduced to less than 650 mm.

The compensation plate changes geometrically a vertical displacement into angular rotations. By increasing the length of the compensation plate, these angular rotations become acceptable. This effect applies vertically as well as laterally (see Fig. 11). By using a compensation plate, the calculated forces for the rail fastenings are not exceeded.

The compensation plate is only installed at the moveable end of the superstructure. The compensation plate‘s ownweight has to be sufficient so as to ensure no lifting of the plate.

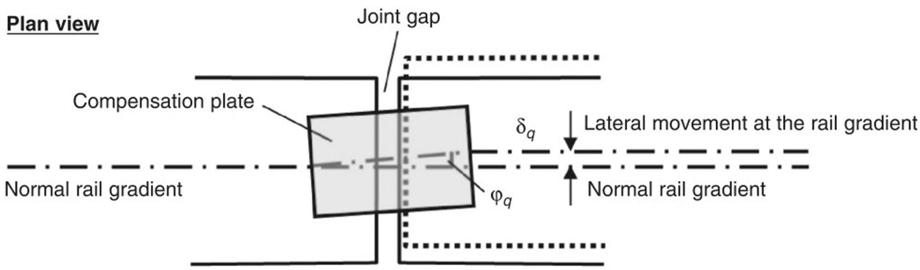

Figure 11. Compensation Plate – plan view.

Compensation plate bearings

All vertical bearings of the compensation plate can move freely in all directions. The vertical displacement of the loaded bearing should not exceed 1 mm, to avoid large tensional forces in the rail fastenings. Furthermore the bearings must be able to sustain dynamic loads.

In order to ensure that the lateral movement of the compensation plate does not exceed ± 1 mm, four bearings of the same design as the vertical bearings are installed horizontally.

The bearings should have the following properties:

– all maintenance of the compensation plate must be quick and possible during traffic (e.g. height adjustment and replacement of bearings)
– the bearings should be maintenance free
– easy replacement of the bearings
– minimal load deformations.

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CHAPTER 13

Structural bearings for high speed railway bridges

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ABSTRACT: High speed railways will be one of the most important challenges for the structural engineers of the third millennium ant the aspect of the structural bearings has an important impact on the design of the structures. The author, currently involved in the design and supply of the structural bearings for the two largest high speed rail projects in the world (Italy and Taiwan), describes the different solutions of the bearing systems utilised in many projects around the world, putting in evidence their impact on the design of the structures. Also the normative aspects, quality assurance and tests are treated.

1 INTRODUCTION

Indeed the construction of railways will be one of the most important issue for the civil engineer in the next decades, world wide.

The European high speed rail system was proposed in 1985 by the EC and includes 23000 km of line, more of half of which to be newly built. According to the programs the European system shall be completed in 2010. At present time the most important sections under construction are in Italy (Novara – Turin, Milan – Bologna, Bologna Florence for a total length of 350 km) but very soon will be started other sections again in Italy but also in Portugal and other European countries.

Outside Europe there are also many thousand km under construction or planned in the near future, specially in Japan, Taiwan and China.

The design speed of the trains, normally 330 km/h, imposes severe geometrical limitation to the track layout in terms of minimum radii of curvature ($R > 7000$ m) and maximum gradient ($< 1.8\%$), so that, when the line crosses highly urbanised areas the ratio of bridges to track length is very high. For instance from Milano to Bologna, a relatively flat area, there are 50 km of bridges on 180 km of track, and from Taipei to Kaoshung in Taiwan there are 250 km of bridges (approximately 8000 spans) on 350 km of tracks.

Bridges are therefore quite often a very relevant part of high speed railways.

Bridges for high speed railways however shall first of all respond to the requirements of the line and not vice versa: as many responsible of the Railways organisation claim: the structure is a service for the rail. In addition to the above mentioned geometrical and stiffness requirements the most stringent one is given from the fact that the rail shall be continuous, avoiding as far as possible the joints on it. This fact is surely due to the comfort (joints of the rail cause noise to the passengers and to the environment) but also to maintenance problems that would be increased by the presence of joints.

Structures like bridges are subjected to length variations that are normally restrained in one point (the fixed bearings) and causes a concentrated movement, proportional to the distance from the fixed point, at the ends of the structure.

The simply supported beam is the preferred static scheme for the railway bridges because, thanks to the presence of the ballast allowing small movements between the rail and the bridge deck, it makes possible the adoption of continuous rails, with great advantage for the maintenance of rails and rolling stock and for the comfort of the passengers. Continuous bridges, requiring expansion joints in the rails, are normally limited to important river crossings or in proximity of the stations.

Therefore the simply supported beams has been adopted in many nations as the main rule, for instance in Italy, Germany, Taiwan, Japan, China, although with some important exceptions like Spain and France.

Even if the static scheme of the bridges is normally very simple, high speed railway bridges are peculiar systems where the superstructure interacts with ballast, tracks and vehicles from one side and with the structural bearings from the other side in a complicated manner, involving strongly non linear effects.

In the paper are described the different static schemes frequently adopted for the railway bridges, the possible bearing systems and their impact on the design and the behaviour of the structures.

2 FREQUENT STATIC SCHEMES ADOPTED FOR THE HIGH SPEED RAILWAY BRIDGES

The adopted static scheme varies in function of the ground morphology but may be easily classified in three categories:

a) When the line passes through a flat country crossing roads and small rivers the preferred scheme is the simply supported beams with spans ranging between 25 and 35 meter. The length of the spans rarely exceeds these values because the structure shall provide a sufficient stiffness in order to limit the flexural deflection due to live load to 1/3000 of the span or less. Normally the bridge structure consist of single or twin box girders of prestressed concrete. When the roads or small rivers to be crossed are frequent, as normally happens in highly urbanised areas, the bridges may become very long as a result of the gradient limitations. A typical example in this sense is the viaduct Modena with a length of 18 km.

   Within the Italian territory the above described static scheme represents nearly the 90% of the entire bridge length of the line.
b) When the line crosses deep valleys a very frequent solution consist of reinforced concrete arches supporting concrete beams with span ranging between 15 to 25 meter.
c) In case of important river crossings, requiring large spans, the preferred solutions are always those minimising the distance between the rail level and the structure intrados, in order to reduce as much as possible the necessary gradient, hence reducing the need of approach bridges or embankments. The structures normally adopted in these cases are:
   – Steel or concrete arches with lower railway and hangers for spans up to 100 meter.
   – Steel trusses with lower railway for spans up to 150 meter
   – Steel or concrete cable stayed bridges for spans over 150 meter.

3 BEARING SYSTEMS FOR THE HIGH SPEED RAILWAY BRIDGES

The bearing system for a structure is the combination of bearings and structural devices which together provide for the necessary movement capability and for transmission of forces.

The selection of the suitable bearing system for a railway bridge shall always be made taking into consideration the following peculiar requirements:

– Horizontal loads due to braking and traction are very high in comparison with the vertical reactions. This require a special consideration on the way this forces are transferred to the substructures. The problem is amplified if also earthquake forces have to be considered.
– The continuous rails are interacting with the structure, specially for the transmission of the longitudinal load. The bearings shall be able to transfer the longitudinal loads with the minimum possible deformation in order to avoid as much as possible an axial reaction from the rail that may cause buckling and misalignment. Elastomeric bearings shall therefore be excluded, unless combined with rigid restrains.

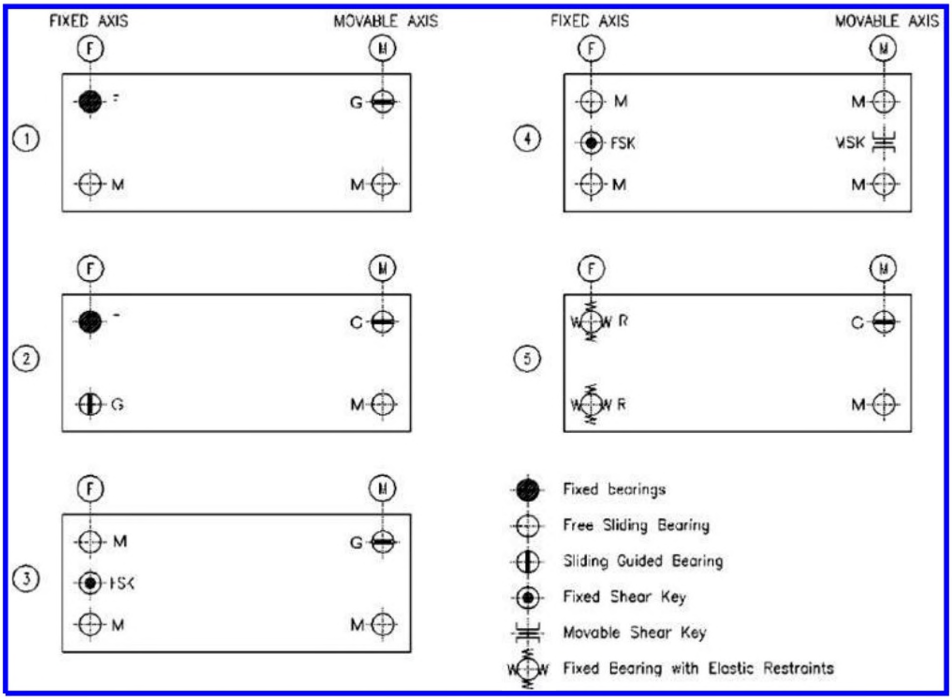

Figure 1. The possible bearing systems for simply supported spans railway bridges.

- In case of earthquake the lateral deflection of the piers may be out of phase, so that the spans may rotate around the vertical axis, requiring the same capability to the bearing system.
- In case of very strong earthquake the bearings may be subjected to uplift forces as a result of two possible situations: lateral earthquake with one train on the track and out of phase lateral deflection of the piers when the bridge deck has a very high torsional stiffness.

The possible bearing systems for simply supported spans railway bridges are summarised in the following figure (see Fig. 1).

Scheme N. 1, although fulfils all the above mentioned requirements, is not very often adopted having a great disadvantage: the horizontal longitudinal force is transferred to the pier out of centre, this means that, in presence of high forces the pier is subjected to torsion.

Scheme 2 has been adopted in some cases because apparently is the cheapest solution but also presents a very negative aspect, due to the fact that different bearings don't react in the same way to horizontal loads.

In a fixed pot bearing for instance the horizontal load will be transferred only after the following mechanical gaps between bearings components have been compensated:

- Gap between lower anchor bolts and bearing plate: normally 0.3 mm
- Gap between pot and piston: normally, in accordance with EN 1337-5, a maximum gap of 0.5 mm is allowed with non metallic seals as required for railway bridges
- Gap between upper anchor bolts and bearing plate: normally 0.3 mm.

In a transversally guided sliding bearing, to the above gaps the following one shall be added:

- Gap between guide bar and sliding plate: normally, in accordance with EN 1337-2, a gap of $1 + L/1000$ mm, where L is the length of the guide is accepted. For the frequent case of bearings with a length of the guide of 1 meter the gap becomes 2 mm (see Fig. 2).

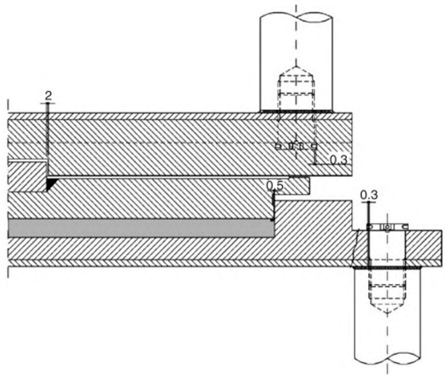

Figure 2. Mechanical plays in a sliding guided bearing.

In conclusion for fixed bearings the total sum of the gaps is 1.1 mm and for the sliding guided is 3.1 mm. Even if this values can be reduced with a very accurate machining (but with a higher cost !), it is clear that the gaps cannot be made equal to zero. Furthermore it cannot be predicted in which direction the gaps will provide a clearance (unless adopting special equalising devices that will be described for scheme 5) and therefore they shall be considered in such a way to give the worst conditions. The result is that one of the two bearings foreseen in the fixed axis of scheme 2 will react first, giving to the pier an eccentric load that will cause a torsion and a bending.

Only when the displacement of the pier in correspondence of the second bearing will fill the existing clearance the second bearing will begin to react to the horizontal loads. The required displacement may be as big as 4.2 mm, that renders this solution applicable only if the piers are very flexible. It shall be considered that, as a consequence of the non simultaneous reaction, each of the two bearings shall be designed for a horizontal force that is higher than 50% of the total and in some cases, when the piers are very rigid, can be near to 100%. In other words the two bearings shall be designed for a horizontal force up to 200% of the actual one with negative impact on the total cost of the bearing system.

In addition this solution don't allow the rotation around the vertical axis and therefore presents a further negative aspect in presence of earthquake.

In conclusion Scheme 2 cannot be recommended, even if it has been adopted in important jobs.

Schemes 3 and 4 are indeed the preferable ones because they transfer the loads to the substructures in a clear and statically determined way, allowing the free movements as required in earthquake conditions. In particular Scheme 3 normally represents the best ratio performance/cost. Scheme 4 is some times required if the bearings cannot resist any horizontal load, like for instance antivibrating elastomeric bearings.

Scheme 5 represents the Italian approach to the problem and consists in the adoption of fixed bearings equipped with an elastic restraint providing a horizontal load reaction increasing with the horizontal deflection. Horizontal load and deflection shall fit a very narrow domain as required by the Italian Railway Authorities and shown in the attached scheme (see Fig. 7). The total deflection is very limited and cannot exceed 1.25 mm. In this way the reactions of two fixed bearings placed on the same fixed axis can be equalised.

For the continuous bridges there will be one fixed axis and several movable axis adopting the same solutions of the above described Schemes, with preference for the schemes 3, 4 and 5.

Quite often the horizontal load to be transferred to the fixed axis is very high and would require excessively resistant piers, specially in presence of earthquake.

A frequent solution in this case is to place on the movable axes Lock-up Devices that allow slow movements due to thermal creep and shrinkage effects with negligible reaction whereas resist rigidly to dynamic forces like braking and earthquake. The use of Lock-up devices however must be

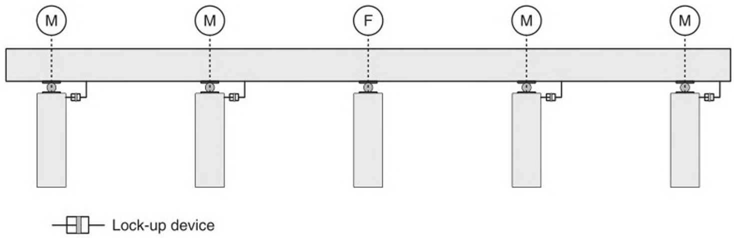

Figure 3. Continuous bridge with lock-up devices.

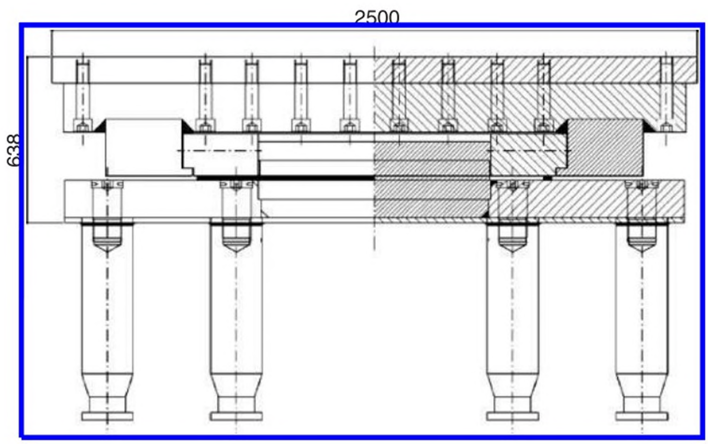

Figure 4. A movable shear key showing the contact between pot and piston aligned with the centre of the sliding surface to minimise the moment.

carefully evaluated bearing in mind their actual performances. First of all it shall be considered that Lock-up devices require a small movement, of the order of 3 mm, to reach the design load. Therefore it is required that the fixed axis is not perfectly rigid but allows a small deflection under horizontal load to allow Lock-up devices to be activated. In addition it shall be noted that the braking forces generated by trains, differently from the forces generated by earthquakes, may remain nearly constant for a period of 30 seconds or more. Therefore it is required that the Lock-up Devices are designed to resist the required force for such a period with negligible or acceptable displacement (see Fig. 3).

4 TYPES OF BEARINGS ADOPTED FOR THE HIGH SPEED RAILWAY BRIDGES

The pot bearing is for sure the most frequently utilised one for high speed railway bridges, with one important exception that is Italy. The reasons for this exception will be explained later.

Pot bearings mainly consist in a rubber disc encased in a steel pot closed by a steel piston. The rubber is subjected to a very high pressure, of the order of 30 MPa and is prevented to be extruded through the gap between pot and piston by a seal that is frequently made of synthetic materials, specially for railway bridges applications. Pot bearings in fact realise the spherical hinge with the minimum possible restoring moment, therefore providing the best conditions for a long service life of the PTFE sliding surfaces. Besides, fixed and sliding guided pot bearings are suitable to transfer high horizontal loads in a reliable way, also in absence of vertical load. The most frequent feature of a sliding guided bearing is shown in Figure 2. If the horizontal load shall be transferred also in absence of vertical load the feature is different and becomes very similar to that of the movable

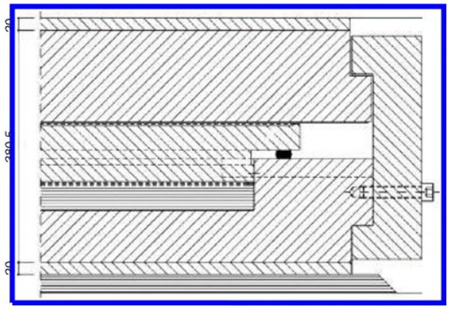

Figure 5. Detail of the antilifting device of a free sliding pot bearings. Small relative rotations are possible at ULS by plastic deformation of the steel.

shear keys that will be described later. In this case the sliding guided pot bearings shall be specially conformed with the guides at the same level of the contact pot-piston in order to avoid undesirable moments. Figure 4 shows this arrangement applied to a movable shear key.

In areas subjected to strong earthquakes, like Taiwan, the bearings may be required to resist uplift, resulting from the combination of a transversal earthquake with the eccentric load of one train or from out of phase displacements of the piers in presence of a bridge deck very rigid to torsion, like for instance prestressed concrete box girders. In that case the bearings shall be provided with an anti-lifting device. Since the uplift force may be considered as a result of a very rare loading combination in ULS conditions, normally it is not required to the anti-lifting device to provide rotation capability. For the Taiwan high speed railway antilifting devices have been realised as shown in Figure 5 by two lateral clamps. Their plastic deformation up to a strain of 0.01 is taken into account in order to accommodate the rotation at ULS.

The most common bearing for the Italian high speed railway however is not the pot bearing but the spherical one. Spherical bearings have the advantage in comparison with pot bearings to allow a much greater rotation. Normally the rotations in railway bridges are very small due to the deflection limitations: a defection of 1/3000 of the span corresponds to a rotation on the bearings axes of 1.3 milliradians where the allowable rotation of pot bearings is normally much greater, of the order of 10 milliradians. In Italy a very large part of the railway bridges are made of prefabricated beams and it is very convenient to accommodate the prefabrication tolerances, normally of the order of 5 to 10 milliradians, utilising part of the rotation capacity of the bearings. In this way it is possible to place the prefabricated beam directly on the bearings avoiding the use of temporary supports. For this reason the Italian Railway authorities selected the use of bearings with rotation capacity of $\pm 3°$ corresponding to ± 52 milliradians, largely sufficient to absorb both flexural rotations and prefabrication tolerances. In this case however it is essential to install the bearings with the sliding surface on the bottom. The sliding surfaces are installed perfectly horizontal and are not affected by the rotation of the bearing due to the prefabrication tolerances. In this way one can be sure that all the sliding surfaces are parallel. The stainless steel of the sliding surfaces must be protected by a suitable dust cover in order to avoid accumulation of dust or debris on it.

The bearing layout normally adopted in the Italian high speed railways is the scheme 5 as described in the previous paragraph. Two fixed bearings are placed on the fixed axis. They are equipped with an elastic restraint that is able to equalise the horizontal loads in the two fixed bearings. In the example of fixed bearing shown in Figure 6 the elastic restraint consists in a steel ring acting between the upper steel plate with concave sliding surface and the lower one with flat sliding surface. The steel ring is stressed by bending and torsion by the horizontal load. The horizontal load deflection plot of the fixed bearing shall be included in the domain defined by the Italian railway authorities shown in Figure 7 and one prototype bearing selected at random shall be tested for every delivery lot. As it may be seen from the diagram the horizontal deflection under load is very small and requires a very accurate machining.

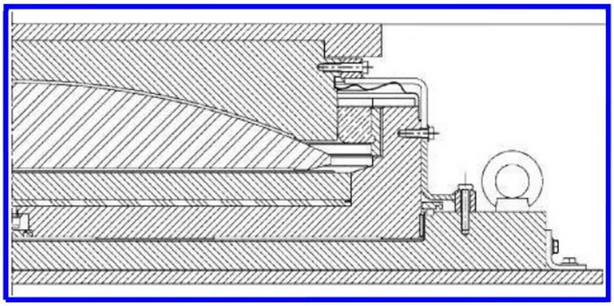

Figure 6. A fixed spherical bearing utilised for the Italian high speed railway. The bearing is equipped with an elastic restraint to equalise the horizontal loads and with a load cell.

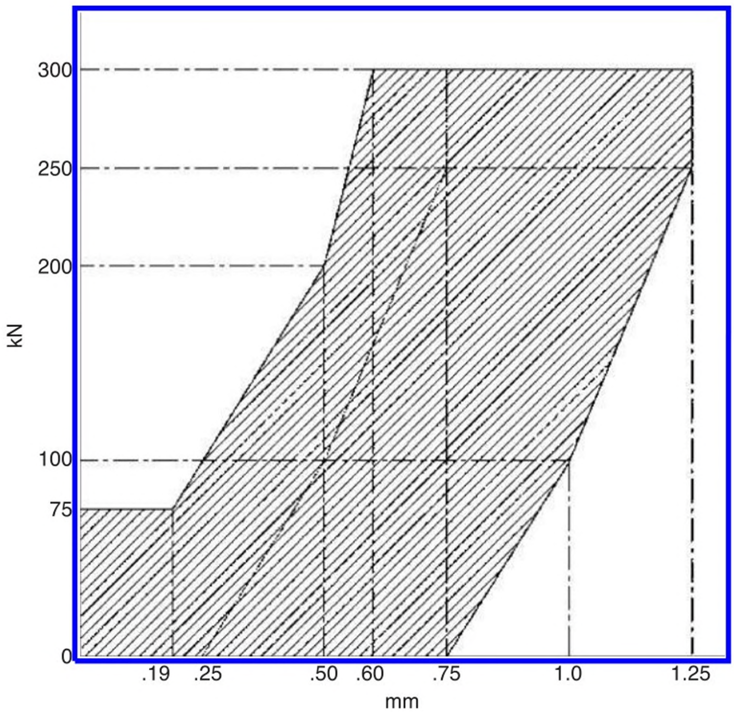

Figure 7. The domain horizontal load – horizontal deflection required for the fixed bearings with elastic restraint.

A few percent of the bearings installed in the Italian high speed railway is required to be equipped with a load cell as in the example shown in Figure 5.

Shear keys are utilised with schemes 3 and 4. They are very similar to pot bearings with the important difference that they don't have a rubber disc since they are not asked to bear a vertical load. The absence of vertical load requires the movable shear key to be conformed as shown in Figure 4, with the contact line between the pot and the piston aligned with the centre of the sliding surface. Both fixed and movable shear keys are free to rotate around the vertical axis thanks to the circular contact between the pot and the piston. The rotation around the vertical axis is specially required in seismic areas to compensate the out of phase movement between the piers.

Shear keys shall be designed to allow small vertical movements to compensate the deflection of the bearings under load without reaction.

5 NORMATIVE ASPECTS

The European Standard on Structural Bearings is now a reality and all partners involved in the construction of a High Speed railway-owners, designers, supervisors, contractors and specialist suppliers- shall be aware of that.

European Standard on Structural Bearings, EN 1337, consist of 11 parts, each one as individual standards. Six of the 11 parts are harmonized standards, so their application is compulsory for every member state of CEN (the 25 nations of the European Community plus Switzerland) due to the binding agreement signed by each of them. The standards will become compulsory after a transition period of 21 months during which the new and the old standards may coexist.

In the following table are listed the parts of the standard, whether they are harmonised or not, when have been approved and when they will become compulsory (the dates are subject to slight variations in the different member states).

It may be seen that for the spherical bearings the application of the European Standard is already compulsory, whilst for all the other types of bearings this will happen in less than two years. It is clears that the use of the old Standards (for instance BS or UNI) could only be admitted for projects that are now under construction. All new projects shall be exclusively based on the relevant parts of EN 1337.

Standard	Title	Harmonised (Y/N)	Approval	Compulsory
EN 1337-1	General Design Rules	N	2001	
EN 1337-2	Sliding Elements	N	10/2001	
EN 1337-3	Elastomeric Bearings	Y	04/2004	01/2006
EN 1337-4	Roller Bearings	Y	02/2004	11/2005
EN 1337-5	Pot Bearings	Y	04/2004	01/2006
EN 1337-6	Rocker Bearings	Y	02/2004	11/2005
EN 1337-7	Spherical and Cylindrical Bearings	Y	10/2001	07/2003
prEN 1337-8	Guide and Restraint Bearings	Y		
EN 1337-9	Protections	N	2001	
EN 1337-10	Inspection and Maintenance	N	2003	
EN 1337-11	Transport, Storage and Installation	N	2003	

Harmonised European Standards imply affixing on the bearings the CE mark.

CE marking is much more than a simple attestation of conformity: is also a Quality Assurance certification much stronger than ISO 9001.

To be authorised to affix the CE marking on the bearings a manufacturer shall be approved by a notified body. Notified bodies are Independent authorities like ministries, universities or laboratories appointed by the CEN. Before giving the approval the notified body will verify the Factory Production Control (FPC) of the manufacturer and perform (or supervise the performance) of all the prototype tests foreseen by the relevant parts of EN 1337.

FPC, that shall be made under the supervision and regular audits from the notified body, includes the following operations:

- The specification and verification of raw materials and constituents
- The controls and routine tests to be performed during the manufacture according to the frequencies specified in the relevant parts of EN 1337

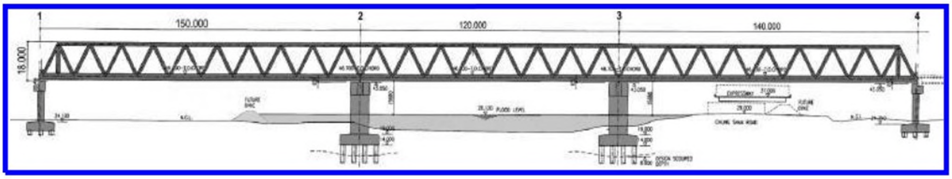

Figure 8. The steel bridge over the river near Taichung station of the Taiwan High Speed Railway.

– The controls and tests to be performed on finished products according to the frequencies specified in the relevant parts of EN 1337.

Under the supervision by the selected notified body the manufacturer shall exercise a permanent Factory Production Control (e.g. a quality management system based on the relevant parts of the ISO 9000 series or otherwise).

The manufacturer is responsible for organising the effective implementation of the Factory production Control.

CE mark represents a certification that the products have been manufactured and tested in accordance with the relevant parts of EN 1337. According to the CEN rules the final user of the bearings cannot require supplementary tests or controls to verify the conformity to the standards.

However the final user may require to the bearings supplementary requirements that may be subjected to further tests or verifications.

The following are two important requirements that are requested by the Italian High Speed Railways Authorities:

– Bearings shall be dielectric. This requirement aims to avoid the propagation in the structures of the so called stray current, connected with the power supply of the train which may cause corrosion effects on the steel reinforcement. Bearings in Italy are required to be intrinsically dielectric (it means without the application of external paintings or insulating materials) with a minimum resistance of 106 Ohm at 1000 V.
– The fixed bearings shall be equipped by an horizontal load equalization system as described at paragraph 4.

6 A FEW OUTSTANDING CASES

Taiwan High Speed Rail with its 8000 spans over a length of 350 km, completely built in 30 months, for sure represents by itself an outstanding case, but is also remarkable for the following aspects:

– Has been the first application, world wide, of the European Standard EN 1337. Although not yet completely approved the standard in it's 1996 draft has been included in the tender documents end therefore became compulsory for the job. It has been applied to the design, manufacture, quality control and testing of around 32000 pot bearing in accordance with EN 1337-5 and 2000 shear keys in accordance with prEN 1337-8.
– Include some exceptional structures that are described here below.

There are three steel bridges crossing the river, near the Taichung station. The most outstanding is that shown in Figure 8. It consists of a continuous steel truss with maximum span of 150 meter and a beam height of 18 meter.

The bearing layout is shown in Figure 9 and slightly differs from the schemes described at paragraph 3. The fixed axis (axis N. 2) is provided with 2 transversally guided bearings resisting the longitudinal horizontal loads and a movable shear key resisting to the transversal horizontal loads.

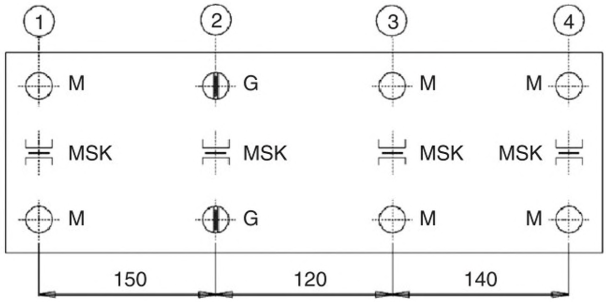

Figure 9. Bearing system of the bridge over the river in Taichung.

Figure 10. Giant shear keys with a design horizontal load of 31000 kN for the steel bridge near Taichung of the Taiwan High Speed Railway.

Since the steel superstructure is relatively flexible the longitudinal horizontal load can be shared in approximately equal parts among the two guided bearings that can also provide sufficient transversal movement capacity to absorb the thermal effects.

For all the other axes of the bridge are foreseen a longitudinally movable shear key and two free sliding bearings allowing longitudinal and transversal movement.

The bearings adopted for this bridge are also absolutely outstanding for what concerns their bearing capacity.

Bearings on axis N. 2 are designed for a vertical load of 82200 kN and a longitudinal horizontal load of 21000 kN. The shear key on the same axis is designed for a transversal horizontal load of 31000 kN. It is shown in Figure 10 and its own weight is of 32 ton.

An other exceptional application of special bearings has been adopted in the Taiwan High Speed Railway.

Part of the railway, for a length of 5 km approximately, crosses an important Hi-tech industrial park in which many industries very sensible to vibrations will be located. To reduce the vibrations in the surrounding environment flexible structural bearing has been adopted, realising the so called mass-spring-system. The mass-spring-system is well known to be the most efficient solution to reduce vibrations and in this case it is particularly performing due to the fact that the mass supported by the flexible bearings consists of the entire bridge superstructure and superimposed dead loads. The vertical stiffness of the bearings has been selected in order to obtain a natural frequency of the superstructure for vertical vibrations of the order of 10 Hz, providing a sufficient gap with the

Figure 11. Antivibrating bearing for the Taiwan High Speed Railway.

Figure 12. The Taiwan High Speed Railway under construction.

natural frequency of the vehicles from one hand and the frequencies of structure-borne sound on the other hand. The bearings with the required vertical stiffness has been developed utilising a very low damping natural rubber compound suitable to minimise the dynamic stiffening effect. They have been designed for the service conditions vertical loads and flexural rotations – in accordance with the European Standard Draft EN 1337-3. The horizontal movements due to creep, shrinkage and thermal effects are allowed, at the movable end of each span, by PTFE and stainless steel sliding surfaces in order to minimise the shear deformation of the rubber. All horizontal loads due to service and earthquake loads are supported by mechanical shear keys totally independent from the rubber bearings. The bearing system reflects scheme 4 as described at paragraph 3, with elastomeric bearings acting as free sliding ones.

An antivibrating bearing installed under the bridge is shown in Figure 11.

Figure 12 shows a section of the Taiwan High Speed Railway under construction.

7 CONCLUSION

High Speed Railway bridges, even if their static scheme may appear very simple, are peculiar structures where the interaction with the ballast and the rail from one side and the bearings from the other side, shall be evaluated carefully. There are several bearing systems that is possible to apply and the designer shall select the right one in function of the performance required. The aim of the paper has been the review of the possible bearings system putting in evidence their performances and the problems related to their utilisation.

The European Standard EN 1337 is the most complete and updated document on structural bearings and for sure will be able to provide the necessary warranty of good performance and quality of these important structural devices.

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CHAPTER 14

Serviceability limit states in relation to the track in railway bridges

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1 INTRODUCTION

The deformations, displacements, and accelerations at the decks of railway bridges which affect the installed track are serviceability limit states for the bridge, but correspond to ultimate limit states for the track and for the circulating vehicles, as these affect their safety.

The engineers dedicated to the design and concept, execution and exploration of railway bridges should not forget at any moment that the bridge must support another structure: the track (with or without ballast), to which the loads acting on the bridge are transmitted and which conditions for its correct and safe functioning have to be assured at all times.

2 DISPLACEMENTS AND DEFORMATIONS

The displacements in relation to the track (that influence its safety and behaviour) which need to be controlled from the moment of conception of the bridge refer to:

- Longitudinal displacements of the deck
- Vertical displacements of the deck
- Vertical rotation of the deck
- Transverse displacement of the deck
- Transverse rotation of the deck
- Bending of the deck.

In this paper are not included the longitudinal displacements resulting from the track-deck interaction phenomena, which enable to analyse the transmission of thermal loads and of the bracking/traction loads between the track and the bridge and which determine the need to install dilation devices in the track, nor those related with passengers comfort.

2.1 *Vertical displacements and rotations*

At the ends of the deck, which are in contact with an abutment (with insignificant vertical movements) or with the end of another deck, the vertical displacements need to be limited. This limitation is associated with the vertical deformation of the rail, in order to avoid excessive bending stresses, with the clamping conditions and vertical flexibility of the rail support (the sleeper for the ballasted track or the slab for the non-ballasted track) and with the possibility of instability of the ballast, as a consequence of this repetitive effect (Fig. 1).

For the case of a ballasted track, in the joints between decks or between the deck and the abutment, the maximum relative vertical displacement δ_v of the platform of the track should be less or equal to 3 mm for the case of tracks with a maximum circulating speed not greater than 160 km/h, and less or equal to 3 mm, if the circulating speed can exceed 160 km/h.

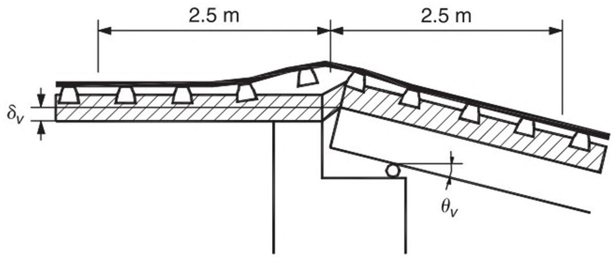


Figure 1. Vertical deformation of the rail at the end of the deck.

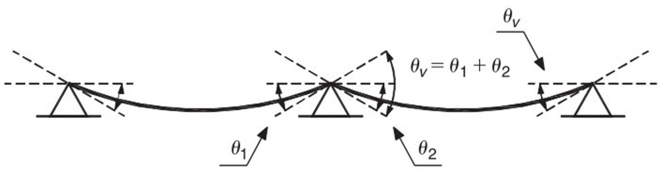


Figure 2. Schematic representation of the rotations at the end of the deck and in the joints between decks.

For the case of a non-ballasted track, specific studies are required for one or both sides of the joints, as a function of the type of rail and the characteristics of the rail support.

This check needs to be made for any type of track, including those with joints.

The calculation of these displacements should be carried out in the axis of each track with the vertical load train for railway traffic (that is, the load train model UIC 71, affected by the corresponding classification coefficient and the impact coefficient) and the thermal loads, whichever is more adverse. For the case of a deck with several tracks, the load train should be considered applied only to a maximum of 2 tracks.

Any discontinuities of the rail (that is, joints, track expansion devices and switches) should be taken into account, by being located at more than 2.5 m of the joint between decks or between deck and abutment, in order to ensure an adequate behaviour against this phenomenon (Fig. 1).

For the same reasons as for the case of vertical displacements, it is necessary to limit the rotations in the vertical plane ϑ_v at the end of the deck in the joints between decks or between deck and abutment (Fig. 1). Using the values of the vertical loads for railway traffic and the thermal loads, whichever is more adverse, the maximum vertical rotation at the end of the deck in the axis of a ballasted track should not exceed the following values (Fig. 2):

- $6.5 \cdot 10^{-3}$ rad (single track) between the deck and the abutment.
- $10 \cdot 10^{-3}$ rad (single track) between consecutive decks.

For the case of multiple tracks, only one track is considered loaded and the limit values (resulting from the hypothesis of coinciding two trains at the bridge) are:

- $3.5 \cdot 10^{-3}$ rad between the deck and the abutment.
- $5 \cdot 10^{-3}$ rad between consecutive decks.

For the case of a non-ballasted track, at one or both sides of the joint, specific studies are necessary, to take into account the characteristics of the fixings of the rail to the slab.

In Figure 3, some construction details are indicated for cases where the prescribed limits are difficult to achieve.

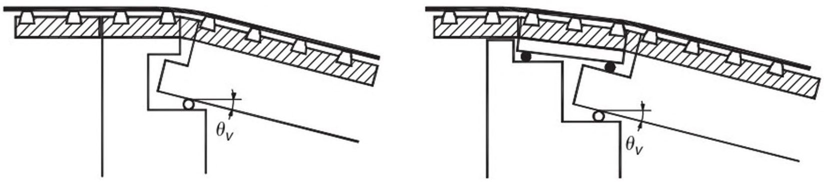


Figure 3. Specific construction details for the case of a non-ballasted track.

Table 1. Maximum angle variation and minimum curvature radius.

		Minimum curvature radius	
Range of speed (km/h)	Maximum angle	Single deck	Multi-deck bridge
$V \leq 120$	0.0035 rad	1.700 m	3.500 m
$120 < V \leq 220$	0.0020 rad	6.000 m	9.500 m
$220 < V$	0.0015 rad	14.000 m	17.500 m

2.2 *Transverse displacements and rotations*

The transverse displacements and rotations of the deck affect the geometry of the track in plan, and its deformation should be limited; since the geometry depends on the speed, these limits also depend on the maximum speed of the railway line.

This condition should be verified with the load due to railway traffic on one track, the wind loads, and the effect of thermal loads. The maximum horizontal deformation δ_h of the deck cannot produce any of following phenomena:

- An angle (with the longitudinal axis) at an end or a joint, greater than the values defined in Table 1. For the case of a joint between consecutive decks, the limit should be applied to the relative angle.
- A radius of horizontal curvature lower than the values indicated in Table 1. The curvature radius can be calculated using the formulae:

$$R = L^2/8\delta_h \tag{1}$$

The horizontal deformation includes the deformation of the deck of the bridge and of the substructure (including columns, piles and foundations). For speeds $V > 120$ km/h and $\alpha > 1$, the load classification coefficient α should be taken as $\alpha = 1$. The limitation for $V = 120$ km/h should be verified independently.

The curvature resulting from 1/R and that of the outline of the track should be compatible with the speed of the railway line.

In order to ensure the continuity of the transverse geometry of the track, the decks and supports should be designed so that the relative transverse displacement, between the end of the deck and the abutment or between ends of consecutive decks, is impeded.

With the purpose of avoiding lateral resonance phenomena in the vehicles, the frequency of first transverse mode shape of the (unloaded) bridge decks should be no less than 1.2 Hz.

To guarantee that the transverse vibrations of the bridges are of small amplitude, the transverse displacement should be limited at any point to 6 mm, under the action of the lateral shock force, taking into account not only the deformation phenomena of the section as well as those due to potential bending or twist. To these effects, the lateral shock force should be combined exclusively with the vertical components of the load due to railway traffic, affected by the corresponding dynamic coefficient or the unloaded train (load: 10 kN/m), whichever results most unfavourable.

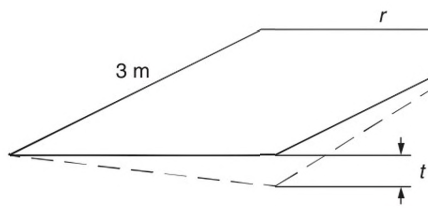


Figure 4. Twist between two cross-sections at a distance of 3 m.

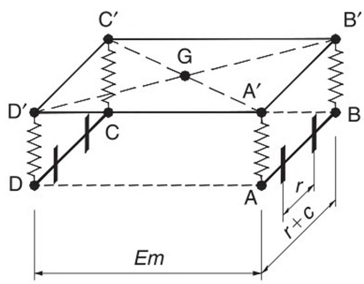


Figure 5. Schematic representation of the suspension of the vehicle.

2.3 *Deck twist*

The twist (t) of the deck of the bridge (Fig. 4) is calculated with the load due to railway traffic.

The maximum twist, measured between two cross-sections at a distance of 3 m, should be no greater than:

$T \leq 4.5\beta$ mm/3 m for $V \leq 120$ km/h
$T \leq 3.0\beta$ mm/3 m for 120 km/h $< V \leq 220$ km/h
$T \leq 1.5\beta$ mm/3 m for $V > 220$ km/h

From which:
$\beta = 1.78r^2/(r+c)^2$, with $c = 0.5$ m.
The limitation for V = 120 km/h should be verified independently.

In Figure 4, r is the separation between wheels; the value of r can be taken as the track gauge plus 65 millimetres.

Unless otherwise stated, the total twist, sum of the potential twist of the track and that corresponding to the actions of permanent and transient loads and of thermal and wind loads, should not exceed 7.5β mm/3 m.

Justification of the formulae which determines the value of β:
As a consequence of the twist of the track, the wheels of the vehicle (bogie) produce variations on the load transmitted to the rail. For its calculation, it is necessary to take into account the characteristics of the suspension.

In Figure 5, the suspension of the vehicle is schematically indicated; the springs have the same stiffness k; the distance between springs is $r + c$; $c/2$ is the distance between the axle of the wheel and the point of support of the spring in the bogie; for railway vehicles, it is sufficiently approximate to consider this distance as constant and that its value is 0.25 m.

If a vertical displacement of value Z is produced at points A and C (Fig. 6), the springs AA' and CC' will extend, causing on points A' and C' a displacement of value mZ, which is identical, by symmetry with respect to G, to the displacement of points B' and D' (since the plane A' B' C' D' is rigid).

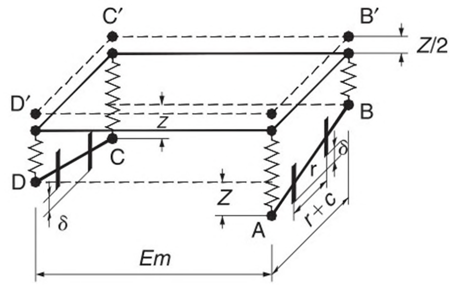


Figure 6. Deformation of the suspension due to a vertical displacement at points A and C.

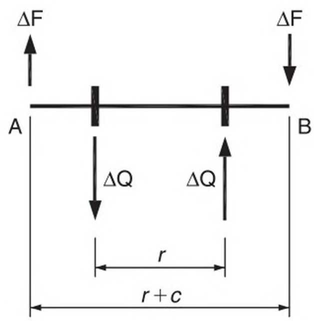


Figure 7. Equilibrium configuration due to a vertical displacement at points A and C.

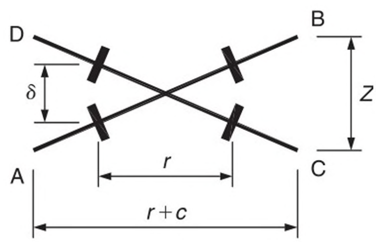


Figure 8. Situation of the axes due to a vertical displacement at points A and C.

The force at the springs AA' and CC' will vary by $\Delta F_1 = k(1-m)Z$ and that of springs BB' and DD' by $\Delta F_2 = k\,m\,Z$, in reverse directions. The equilibrium of the system requires that, in absolute values, $\Delta F_1 = \Delta F_2$, that is:

$$k(1-m)Z = k\,m\,Z \Leftrightarrow m = 1/2 \tag{2}$$

Hence, the absolute value of the variation of such force will be $\Delta F = 0.5\,k\,Z$.

Considering an axis in the new equilibrium situation and taking in account that the respective angles are very small, the following can be stated according to Figure 7:

$$r\Delta Q = (r+c)\Delta F \tag{3}$$

The situation of the axes is illustrated in Figure 8, from which the following can be deduced:

$$Z/\delta = (r+c)/r \Leftrightarrow Z = \delta(r+c)/r \tag{4}$$

The twist corresponding to the wheel base Em, which is the distance between the contact point with the rail of the wheel associated with A and the plane defined by the contact points with the rail of the wheels associated with B, C and D, will have the value of 2δ. Therefore, a twist of 2δ will correspond to an unload of the wheels A and $C$of the value:

$$\Delta Q = (r + c)/r \cdot \Delta F = 0.5k\delta(r + c)^2/r^2 \tag{5}$$

Safety towards derailment will be the same for the same value of ΔQ from which, taking a twist of $2\delta_0$ associated with r_0 as reference and considering k and c as constant along the width of the track, the level of safety will be maintained if:

$$k\delta_0(r_0 + c)^2/r_0^2 = k\delta(r + c)^2/r^2 \tag{6}$$

That is, to say:

$$\delta = \delta_0(r_0 + c)^2/r_0^2 \cdot r^2/(r + c)^2 \tag{7}$$

Taking $r_0 = 1.5$ m as reference, results the following:

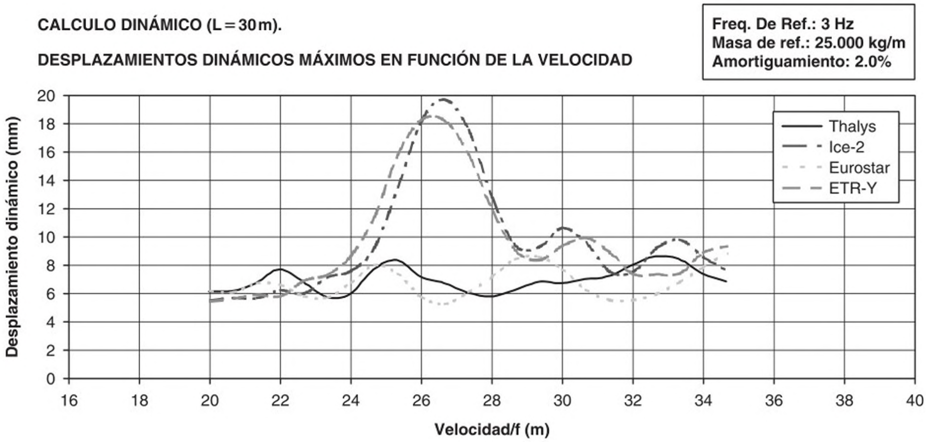
$$\beta = 2\delta/2\delta_0 = (2/1.5)^2 r^2/(r + c)^2 = 1.78r^2/(r + c)^2 \tag{8}$$

3 VERTICAL ACCELERATIONS OF THE DECK

After the first European high speed railway line (Paris-Lyon) began operating, the appearance of disturbances in the ballast layer was detected at certain points, which have caused the instability of the track and the deterioration of its geometry, with evident risks to the circulating vehicles. A study on this phenomenon has shown that it is associated with vertical accelerations of the deck, produced by the passing of trains at specific speeds (resonant speeds which, as a consequence, do not have to be the maximum speeds) by reaching values of the order of 0.7 g to 0.8 g (g: acceleration of the gravity, 9.81 m/s^2). In turn, bridges have been designed according to the UIC 71 load scheme and the rules of the UIC 776-1 standard. Moreover, when fissuring starts to propagate in concrete bridges, the stiffness and the mode shape frequency of the deck are reduced as well as the corresponding resonance speed; for example, a reduction on the resonance speed has been observed, from 287 km/h (for a new bridge) to 250 km/h for the same bridge, some time after its opening to service.

Analysing this question from the theoretical and experimental point of views, the following has been observed:

a) The ballast, when subjected to vertical accelerations of the order of 0.7 g to 0.8 g, experiences a phenomenon similar to liquefaction which causes it to lose its self-supporting strength and produces degradation in the levelling and linearity of the track.
b) The placement of an elastic ballast mat between the deck and the ballast may amplify the accelerations at the ballast, hence aggravating this phenomenon. This important conclusion has been obtained from test results commissioned by the ERRI D 214 Committee to the "Constructions and factory works" division of the Federal Institute of Research and Testing Materials (BAM) of Berlin, who carried out the works in cooperation with DB AG (German railways). Several ballast mats of different characteristics were tested. Quantitatively, the amplification is of the order of 40%.
c) Accelerations of magnitude g, even in non-ballasted decks, may produce a reduction of the wheel-rail contact forces Q (inclusively causing the loss of contact) down to unacceptable limits.
d) The regular and repetitive distribution of the axes of the high speed trains may lead (at certain speeds) to situations of resonance of the decks with important amplifications both in the deflections and in the vertical accelerations, as illustrated in Figure 9.



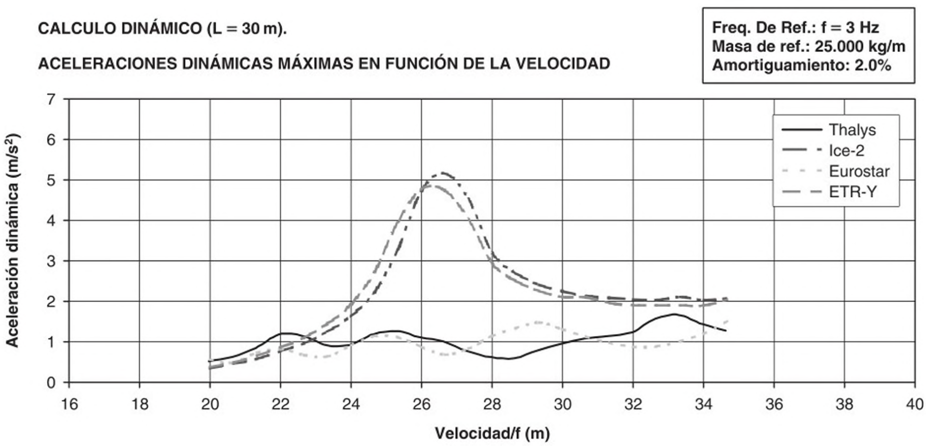


Figure 9. Amplifications due to regular and repetitive distribution of the axes of the high speed trains.

e) From the group of real characteristic trains that served to define the UIC 71 load model (Figure 10) and the applicable dynamic coefficients, only the number 5 train (Turbotrain for V=300 km/h) reaches the indicated speed, and the train consists of two vehicles, with a total length of 38.4 m and 8 loads of 170 kN, facing the modern high speed trains, with a possibility of extending to 400 m in length (in accordance with the TSI - Technical Specifications for Interoperability) and 10 000 kN of total load, circulating at speeds greater than 300 km/h.

As a consequence, it is necessary to analyse the behaviour of the railway structures which have to support high speed circulating vehicles, in terms of its design, ensuring that:

1. The vertical acceleration, at decks of ballasted tracks, is not greater than the value of 0.35 g (factor of safety of 2) for the range of frequencies up to 30 Hz, or up to the double of its first mode shape frequency (whichever is greater), with the track loaded at its most adverse situation.
2. The vertical acceleration, at decks of non-ballasted tracks, is not greater than the value of 0.5 g (factor of safety of 2) for the range of frequencies up to 30 Hz, or up to the double of its first mode shape frequency (whichever is greater), with the track loaded at its most adverse situation.

These checks have to be made for the 10 trains which compose the High Speed Load Model-A (HSLM-A) defined by the ERRI D 214 Committee, recognized by the Eurocodes, TSI's and the new Spanish standard (Fig. 11). The HSLM-A train should cover the effects of real trains, present and future (Fig.12).

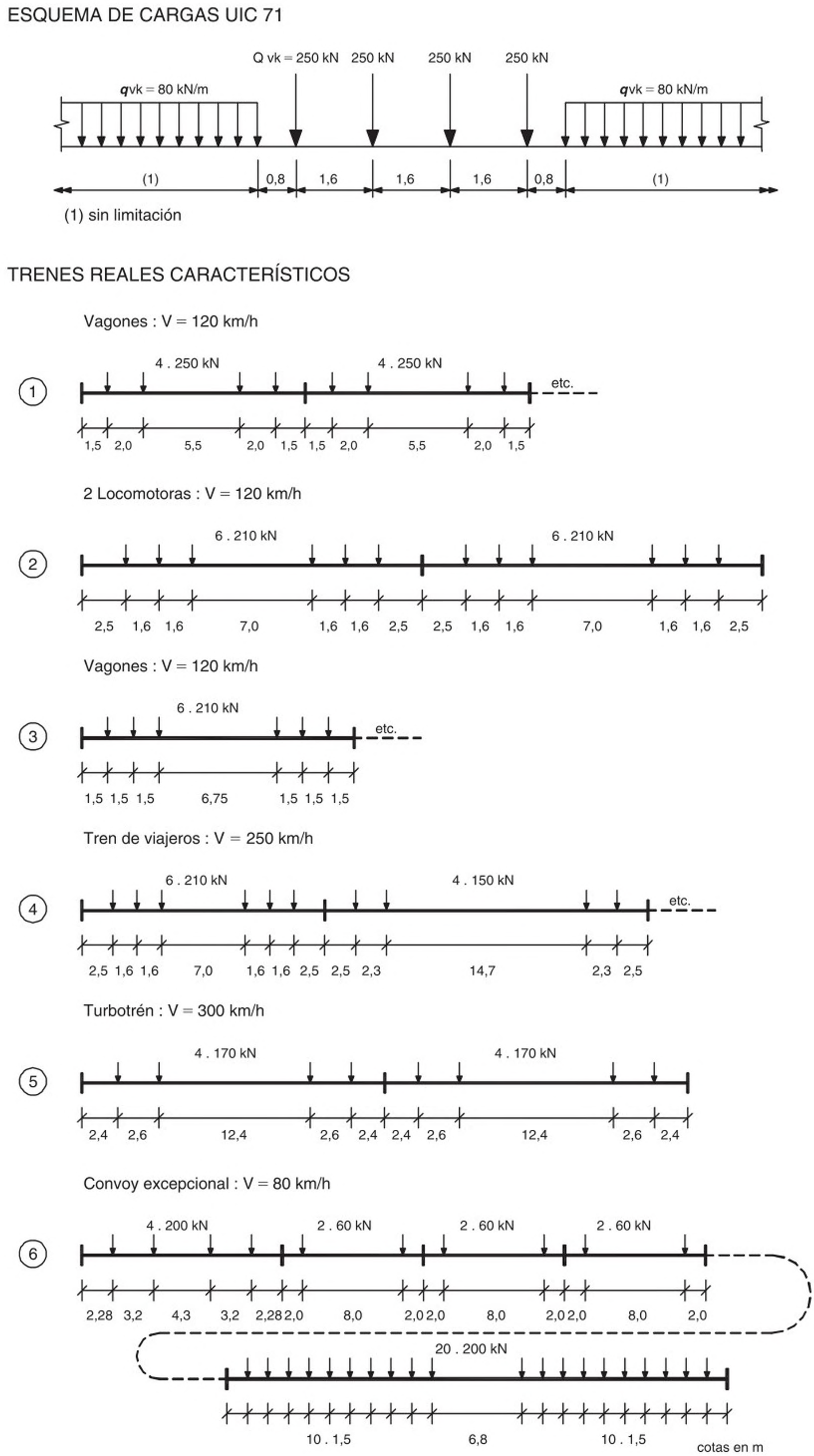
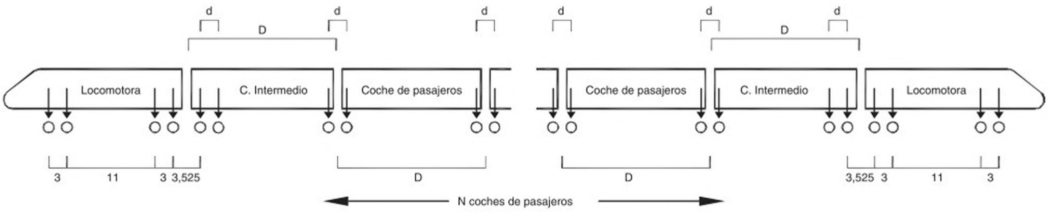
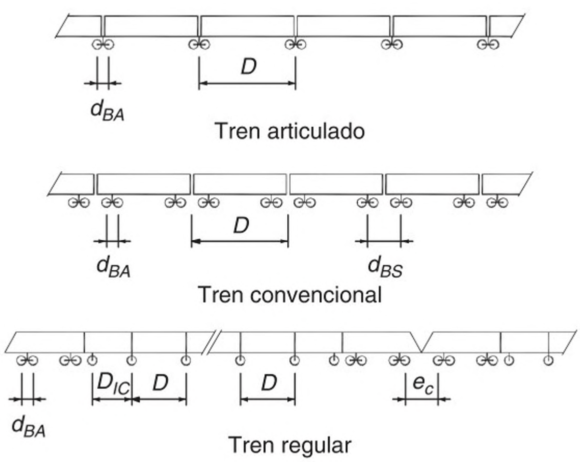


Figure 10. Group of real trains that served to define the UIC 71 load model.



Train	Number of passengers carriages	Length of carriages	Distance between axles of a bogie	Nominal load per axle
	N	D [m]	d [m]	P [kN]
A1	18	18	2.0	170
A2	17	19	3.5	200
A3	16	20	2.0	180
A4	15	21	3.0	190
A5	14	22	2.0	170
A6	13	23	2.0	180
A7	13	24	2.0	190
A8	12	25	2.5	190
A9	11	26	2.0	210
A10	11	27	2.0	210

Figure 11. Definition of the HSLM-A.



Type of train	P [kN]	D [m]	D_{IC} [m]	e_C [m]
Articulated	170	$18 \leq D \leq 27$	–	–
Conventional	Min (170, P_C) (*)	$18 \leq D \leq 27$	–	–
Regular	170	$10 \leq D \leq 14$	$8 \leq D_{IC} \leq 11$	$7 \leq e_C \leq 10$

$(^*)P_C = 0.5P_{TDU\text{-}A}\cos(\pi d_{TDU\text{-}A}/D_{TDU\text{-}A})/[\cos(\pi d_{BS}/D)\cdot\cos(\pi d_{BA}/D)]$

$P_{TDU\text{-}A}$, $d_{TDU\text{-}A}$ y $D_{TDU\text{-}A}$ are the values corresponding to HSLM-A.

Figure 12. Characteristics of present and future real trains.

The dynamic calculation of accelerations is usually made under the hypothesis of a track and wheels without any irregularities, from which the maximum accelerations in the deck or, more precisely in the elements in contact with the track, are obtained. In order to quantify the effects of the irregularities of the track (and the wheels) it is only necessary to multiply the maximum acceleration calculated with $(1+\varphi'')$ or $(1+0.5\varphi'')$ respectively. It is worthwhile reminding that $\varphi'' = v/22 \cdot [0.56e^{-(L_\phi/10)^2} + 0.5(L_\phi n_0/80 - 1)e^{-(L_\phi/20)^2}]$, where v is the speed in m/s, under the condition that if $v > 22$ the value of $v = 22$ should be taken, and that $\varphi'' = 0$ if φ'' results negative.

Given the imprecisions on the calculations and of the parameters – stiffness, damping and mode shape frequencies of the bridge and its possible evolution –, the range of speeds to consider should be comprised within 220 km/h (when considering the load train established in the new Spanish standard, for track widths of 1435 mm and 1668 mm) and 1.2 times the maximum speed of the railway line.

4 CONCLUSIONS

1. In the design of railway bridges, it is necessary to verify the serviceability limit states corresponding to:
   - Axial displacements of the deck.
   - Vertical displacements and rotations of the deck.
   - Transverse displacements and rotations of the deck.
   - Twist of the deck.

   These checks can be made with static calculations, once the applicable impact coefficient has been determined.
2. Recent experiments (and theory) demonstrate that, for the case of high speeds, resonance situations can appear in railway bridges.
3. Traditional impact coefficients do not cover the resonant amplifications of the accelerations in the decks
4. It is necessary to ensure that the maximum accelerations that affect the track do not exceed the values of 0.35 g (ballasted track) or 0.5 g (non-ballasted track). For this purpose, dynamic calculations are indispensable.
5. In the event of non-conformity with the prescribed limit values, the economic repercussions may be very high, eventually leading to speed limitations (with negative impact on the line exploration) or to modifications or even replacement of bridge decks.

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CHAPTER 15

Differences in designing high-speed railway bridges and highway bridges

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ABSTRACT: Live Loads of High Speed Railways Bridges are quite larger than those used for the design of Highway Bridges, resulting in important differences between the design criteria used in each case. The aim of this paper is to show to experimented designers of highway bridges the differences between the design criteria of both kinds of bridges, particularly from the functional, morphological and structural point of view. Decks, piers, abutments, and structural bearings will be considered.

1 REVIEW OF THE SPECIAL CHARACTERISTICS OF HIGH-SPEED RAILWAY BRIDGES

1.1 *Intensity of vertical loads*

Figure 1 shows a schematic diagram of two cross sections, one of a continuous highway bridge with a 49 m central span, and another of a high-speed railway bridge, also continuous, with a typical span of 50 m. In the first case, the total load amounts to approximately 300 kN/m, while the total load for the railway bridge is around 660 kN/m. Thus, the total load for a railway bridge is approximately double that for a highway bridge. Furthermore, if we compare the ratio of the sum of permanent loads plus live loads for each type of bridge, the resulting value is almost four. This

Action	kN (m)
g1	185
g2	45
q + Q	70
TOTAL	**300**

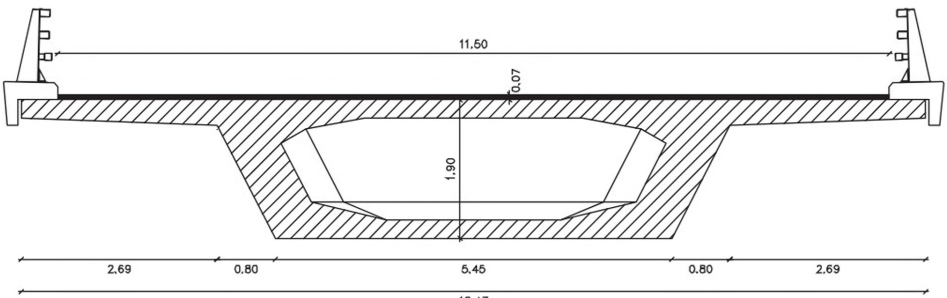

Figure 1a. Typical cross section of a continuous road bridge. $L_{max} = 49$ m.

Action	kN (m)
g1	230
g2	193
q + Q	240
TOTAL	**663**

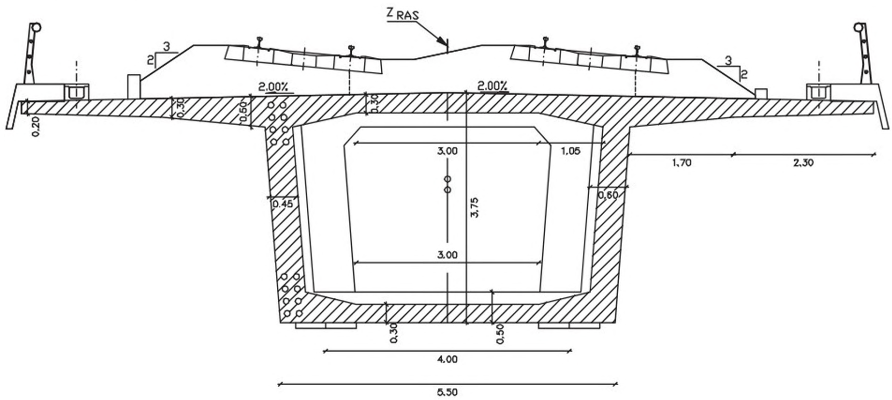

Figure 1b. Typical cross section of a continuous railway bridge. $L_{TIP} = 50$ m.

gives us a preliminary indication of the ratio of depth between the two types of bridge and the type of cross section, in which, for prestressed railway bridges, the efficiency will be important.

1.2 *Loads positioning*

The second factor to be considered is the positioning of loads on the cross section. While live loads on a highway bridge may occur at any point of the cross section, on a railway bridge the position of the rails is fixed, except in the case of bridges located at the exit from a station, where there may be switches. This also determines the position of the webs in the cross section, which will perform the task of carrying the loads, by shear, to the supports.

1.3 *Fatigue*

The third item to be considered is the fact that loads are almost always maximum and that they are repetitive. Loads are almost always maximum because all trains are pulled by locomotives, and they are so repetitive that they can give rise to fatigue.

In fact, if on local lines we estimate a frequency of 1 train every 5 minutes over two intervals of 4 hours, 1 train every 10 minutes over an interval of 10 hours, and no traffic during an interval of 6 hours, this amounts to 156 trains daily, or some 57.000 trains yearly. The figure of two million cycles, the limit of endurance, is reached after 35 years of service, approximately one third of the structure's service life. Consideration of the limit state of fatigue is therefore indispensable.

1.4 *Dynamic effects*

Another characteristic of railway bridges is found in dynamic effects, which are manifest in two phenomena. On the one hand, there is the effect of impact as an increase of static effects owing to the movement of rolling-stock, interference of their suspension systems with the rails and attachment to the structure itself. This effect is particularly important at the wheel-rail interface and decreases drastically in railways with very good maintenance. High-speed railways can be affected by another phenomenon: the resonance of the structure. If the frequency of entry of the axles of the wagons into the structure, i.e. the excitation frequency, coincides with the primary vibration frequency of the structure, its deflections are gradually increased as the train passes over it, giving rise not only to greater stresses, but also to vertical accelerations of the deck, which, if they exceed a given value (0.35 g), can cause the wheels to leave the rails and the ballast to be shaken free from the deck, with the risk of detachment of the rails.

1.5 *Braking and start-up forces*

With highway bridges, many standards estimate braking forces at one twentieth of the total live load, limiting its maximum value, even in long bridges, to around 720 kN. In a double-track railway bridge, the EC-1 standard requires the braking of a 300 m train and the start-up of another train considered simultaneously. For a maximum value for wagon/rail friction of 20 kN/m, braking force reaches 6000 kN and start-up force reaches 1000 kN, given a locomotive length of 30 m. For bridges of over 300 m in length, this means a horizontal force that is 10 times greater in railway bridges than in highway bridges, a force that, when added to the force of friction on the bean, attains values in excess of the design specifications normally applied to fixed bearings.

It is basically these five factors, namely the magnitude of the vertical load, the position of live loads on the cross section, the repetitive and dynamic nature of those live loads and the considerable horizontal forces, that will constitute the differences in morphology and design between railway and highway bridges.

2 PLATFORM AND SUPERSTRUCTURE EQUIPMENT

The basic parameter, namely the separation between track centres, is determined by the planned speed for the line, so that the overpressures arising during the crossing of trains does not cause discomfort for passengers. For a planned speed of 300 km/h, the separation used at present is 4.70 m. The other criteria, along with the aforementioned one, are shown in Figure 2, where we see that the ballast occupies a total width of 10.10 m, that there is a continuous channel on either side for signalling and interlocking, that the anchor blocks of the catenary posts are located outside the through and do not impinge on it, and lastly, that there are two service footways measuring 0.70 m in width, for a total platform width of 14.00 m.

3 DECK CONFIGURATION OF CONVENTIONAL BRIDGES

3.1 *Minimum stiffness*

In Section 1, we referred to the disturbance of ballast by the passage of high-speed trains if accelerations of 0.35 g are attained. Consequently, and with the further aim of controlling certain dynamic stresses and movements, the Spanish high-speed railway authorities require for all such bridges a systematic study of their dynamic response modelling the passage of seven standard trains, or the envelope of all seven, from a passage speed of 220 km/h up to 20% above the maximum speed for the stretch, which is normally $1.20 \times 350 = 420$ km/h. The designer's objective then consists of making the primary fundamental frequency of the deck different from the excitation frequency produced by the axles of the wagons, to avoid resonance. With long, continuous bridges, this is not a difficult task, since the critical excitation of a span is damped by movements induced by the presence of loads on continuous spans, but it is very important in bridges with short spans, which require a minimum stiffness to make the response frequencies different from the excitation frequency.

The reference values given in appendix B [1] can be used as a previous control of frequency range and minimum linear mass that should be accomplished according with the span length.

Furthermore, it is important to control the following movements in order to ensure the comfort of passengers and the preservation of the geometry of the track.

i) Torsional rotation of the deck
ii) Maximum rotation in supports

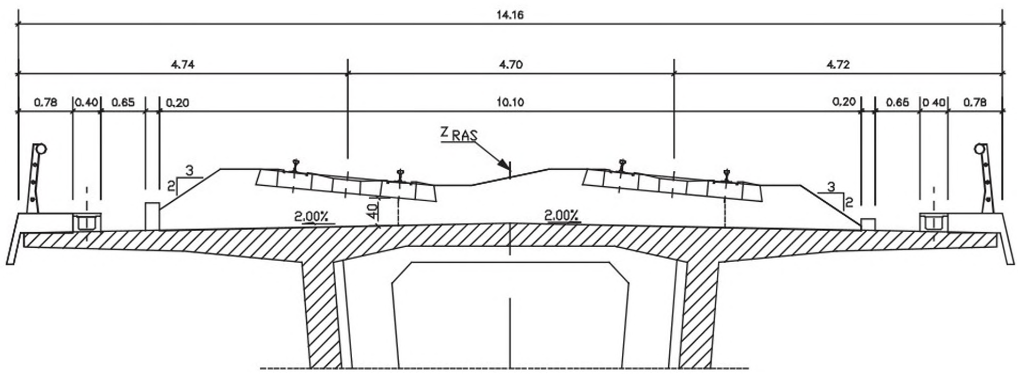

Figure 2. Typical platform and equipment for HSR Spanish bridges.

Table 1. Torsional rotation of the deck.

Speed (km/h)	Maximum vertical displacement in 3 m length (see Fig. 3)
$V \leq 120$	$T \leq 4.50\beta$
$120 < V \leq 220$	$T \leq 3.00\beta$
$V > 220$	$T \leq 1.50\beta$
$\beta = 1.78\ r^2/(r + c)^2$; ; $c = 0.50$ m	
$r = 1.435 + 0.065$ (m)	
Maximum torsional rotation: (SW + DL + LL + Wind + Temp)	
$t \leq\ 7.5\beta$ (mm)	

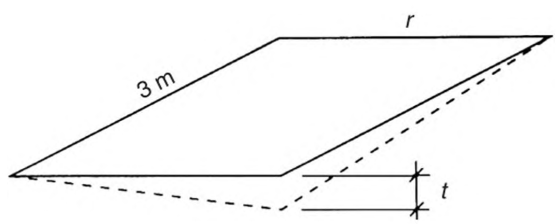

Figure 3. Maximum vertical displacement "t" in three meters length.

Table 2. Maximum flexural rotation in ballasted tracks.

Situation	Traffic	One track	Double or multiple track (only 1 loaded track)
Transition	Live loads	6.5×10^{-3} rad	3.5×10^{-3} rad
Deck/Abutment	Real traffic (HSR)		2×10^{-3}/h*rad
θ	$V > 220$ Km/h		
Distortion between two	Live loads	10×10^{-3} rad	5×10^{-3} rad
consecutive decks	Real traffic		
$\theta_1 + \theta_2$	$V > 220$ Km/h		4×10^{-3}/h* rad

*h(m): distance between rail top surface and rotation axis of bearing.

iii) Horizontal displacements of the deck. Table [3]
iv) Passengers comfort (I): Accelerations
v) Passengers comfort (II) Deflections. Figures 4a and 4b.

Next Figures, 5 to 7, show some experimental and theoretical results obtained with several HSR Spanish bridges.

Table 3. Horizontal displacements of the deck. Railway traffic + Wind + Temperature.

Speed Range Km/h	Vertical rotation axis θ max(rad.)	Maximum Radius, R_{max}, of horizontal deformed shape (m)	
		1 deck	Several decks
$V \leq 120$	3.5×10^3 rad	1700	3500
$120 < V \leq 220$	20×10^{-3} rad	6000	9500
$V > 220$	15×10^{-3} rad	14000	17500

$R_{max} = L^2/8\delta_{H,max}$.
δ_H = Horizontal maximum displacement. Foundation, piers and deck movements must be included.
L = Total deck length for continuous bridges or span length for simply supported ones.

Table 4. Maximum acceleration inside the wagons.

Confort level	Accelerations (ms^{-2})
High	1.00
Good	1.30
Acceptable	2.00

3.2 *Longitudinal schemes*

For short span underpasses (5–10 m), reinforced concrete frames or portals are used with a sufficient depth in vertical members and decks to avoid resonance problems. Underpasses with longer spans are designed with simply supported schemes with a slenderness of h/L <> 1/11÷1/13, using prefabricated pre-stressed concrete u-beams for spans of up to 30 m. For longer viaducts and bridges, the scheme can be a succession of simply supported beams or a continuous deck.

The solution for the succession of simply supported spans must be of considerable depth in order to ensure the minimum stiffness mentioned earlier, and it must be supported on short, heavy piers, since, owing to the two lines of support per pier, the difference in reactions gives rise to longitudinal moments in the pier that cause it to rotate, with the relative rotation between decks increasing, causing problems for preserving the geometry of the track and passenger comfort. In addition, and this subject will be dealt with more comprehensively in the section on bearing devices, it is advisable for decks to be linked, in order to transmit braking forces to a fixed point and at the same time allow use of long sections of welded rail, which are most efficient at providing passenger comfort. Figures 8 and 9 show two examples of solutions, one using short spans, prefabricated beams and short piers (bridge over the Balisa gully, Madrid–Valladolid high-speed rail line), and the other using medium spans, on-site construction and tall piers (Fulda viaduct, German HS Line).

The use of a continuous deck is almost always preferable. Such continuity provides greater stiffness, more "noise" in the dynamic response and therefore fewer resonance problems, and the possibility of using taller piers, since the deck rests on them along a single, centred line of support and the vertical reactions of the deck do not give rise to any rotation, these therefore being less critical solutions in terms of restrictions on preservation of the geometry of the tracks. They involve the problem, which is more theoretical than actual, that in the event of replacement of decks by means of crosswise ripping, as the Germans have required on some lines, the number of provisional supports and hydraulic devices is multiplied. These solutions can be used for lengths of up to 700 m, owing to problems of absorption of movements of devices for controlling expansion of the rails, with constant depths, spans of up to 70 m [4], although the normal range is 45 to 55 m, and slenderness of between one fourteenth and one seventeenth of the span, depending on the construction method. Where prefabricated pre-stressed concrete beams are used, spans can reach 38 m.

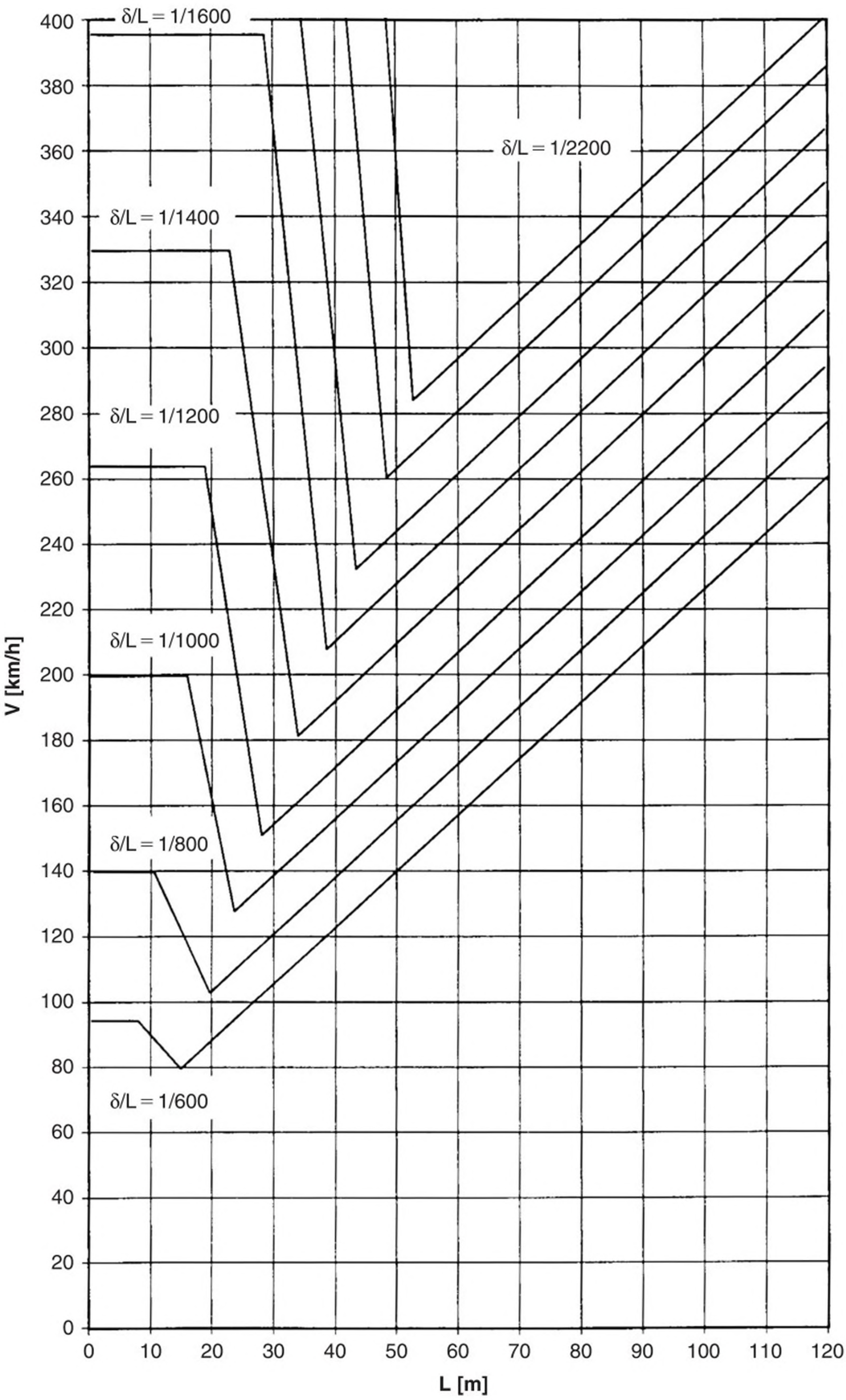

Figure 4a. Limit values for maximum vertical deflections for passengers comfort according to speed and span length. Simply supported spans [1].

3.3 *Cross sections*

For frames and portals up to spans of 10 m, the cross section used is solid slab, with structural problem resolved by means of reinforced concrete. For underpasses, small bridges or short viaducts, with spans ranging from 10 to 30 m and built on-site, the recommended cross section is that of

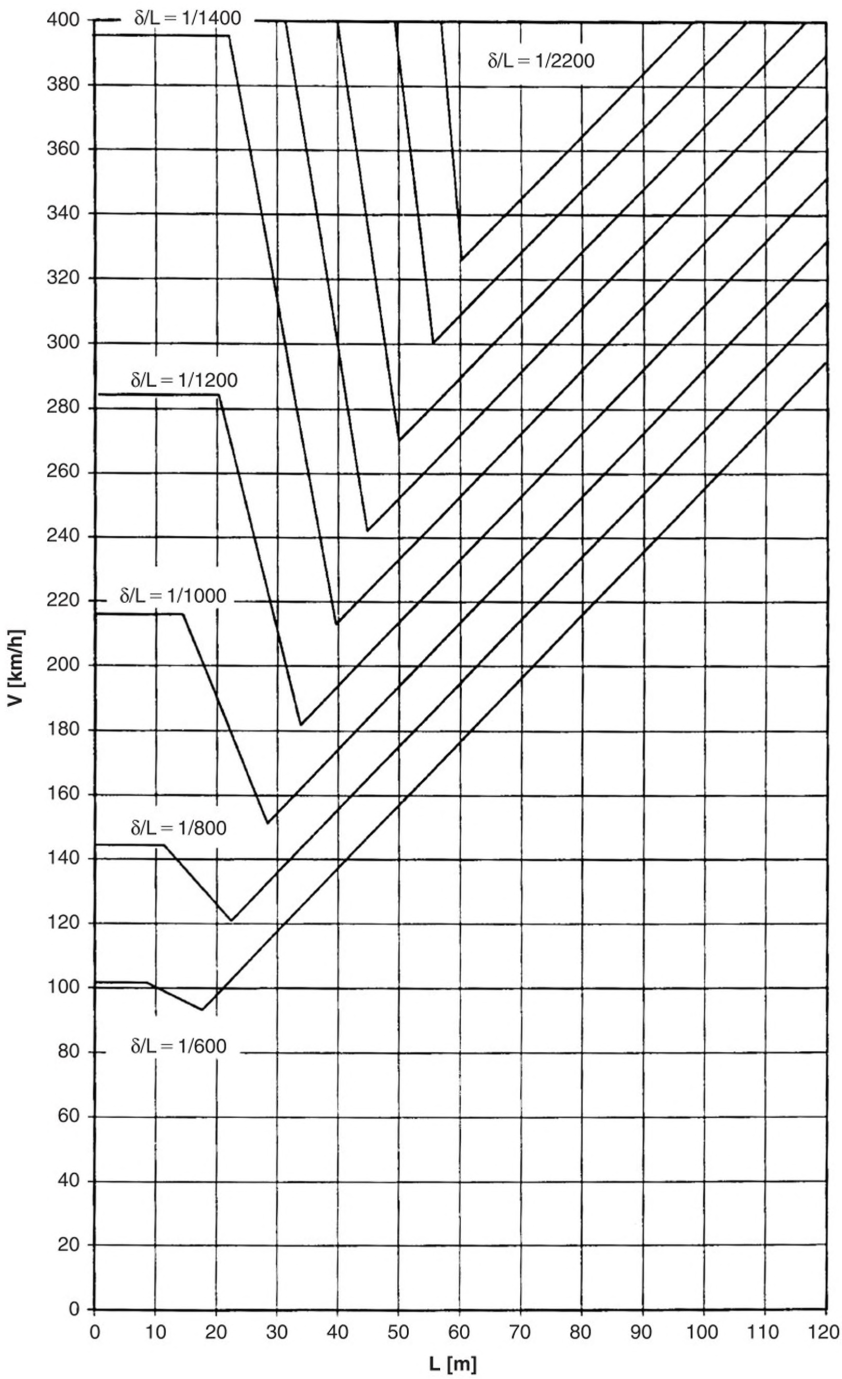

Figure 4b. Limit values for maximum vertical deflections for passengers comfort according to speed and span length. Continuous decks [1].

voided slab with a central core for the tracks and lateral cantilevers to complete the width, Figure 10. The voids should be circular, to avoid transverse bending, and the minimum thickness of concrete over the voids should be around 25 cm. Solutions using prefabricated u-beams work well between 10 and 30 m in simply supported schemes and up to 38 m in continuous schemes, and they can be laid out either separately or abutting to give the appearance of a single-cell box. Figures 11 and 12.

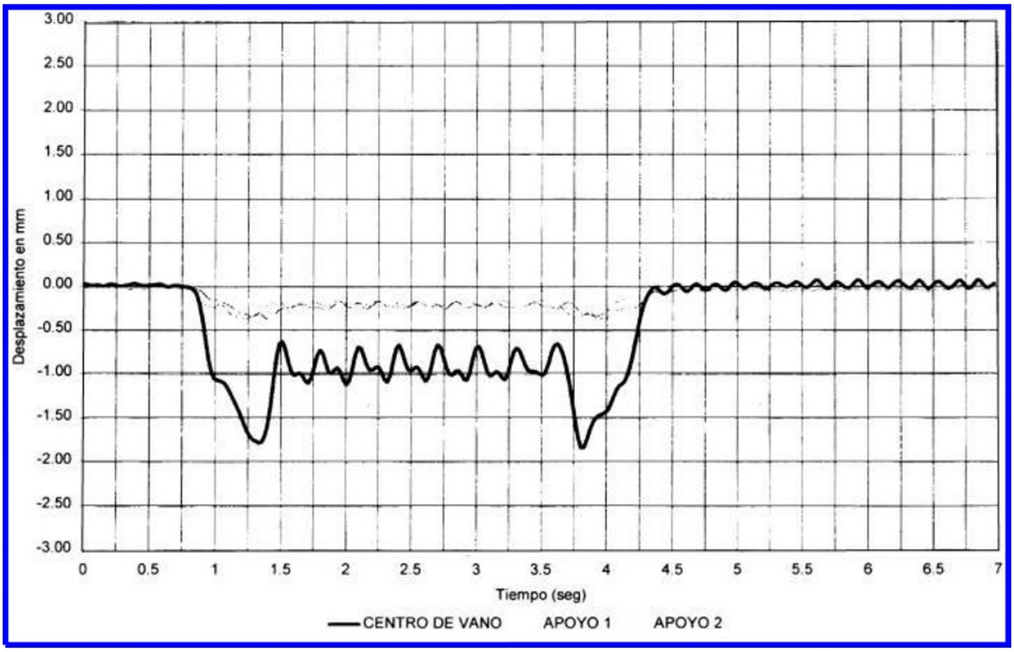

Figure 5. Ribera viaduct, HS Line Madrid–Seville. Experimental deflections versus time curve when crossing AVE train at 200 km /h [2].

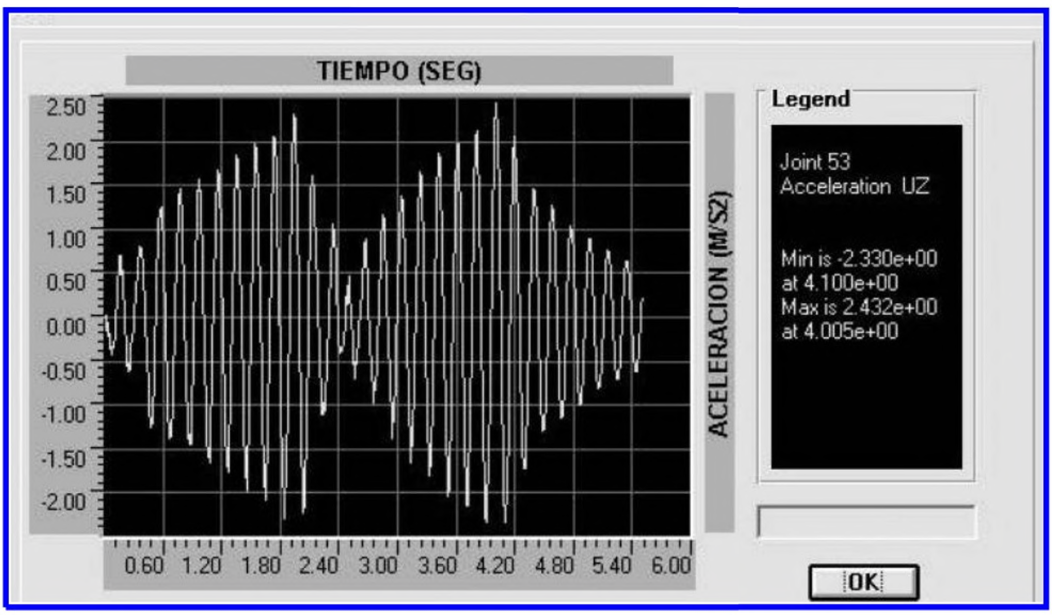

Figure 6. Hontoria viaducte. HS Line Madrid–Valladolid. Theoretical vertical acceleration versus time curve at midspan when crossing AVE train at 350 km/h [3].

For longer spans, between 40 and 70 m, the cross section is always that of a single-cell box of constant depth with the slenderness values mentioned above, which, as noted, are between one fourteenth and one seventeenth of the span, Figure 1b.

All types of cross section must be laid out so that the outer rail is no more than 50 cm outside the face of the web. This prevents the cantilevers from influencing the dynamic response (deflections, speeds and accelerations) perceived by the trains, since otherwise those parameters would be greatly increased.

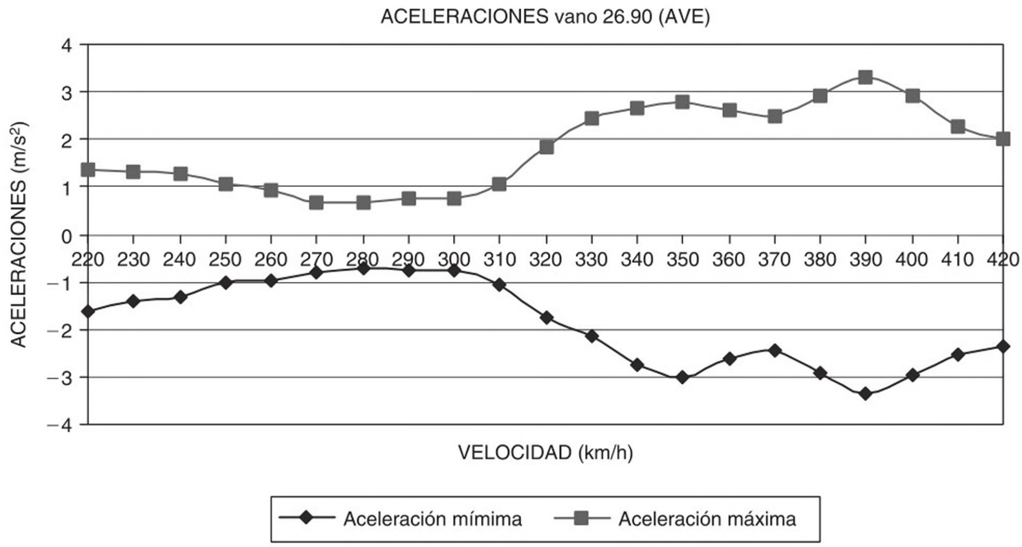

Figure 7. Hontoria viaducte. HS Line Madrid–Valladolid. Theoretical maximum and minimum acceleration versus speed curve when crossing AVE train [3].

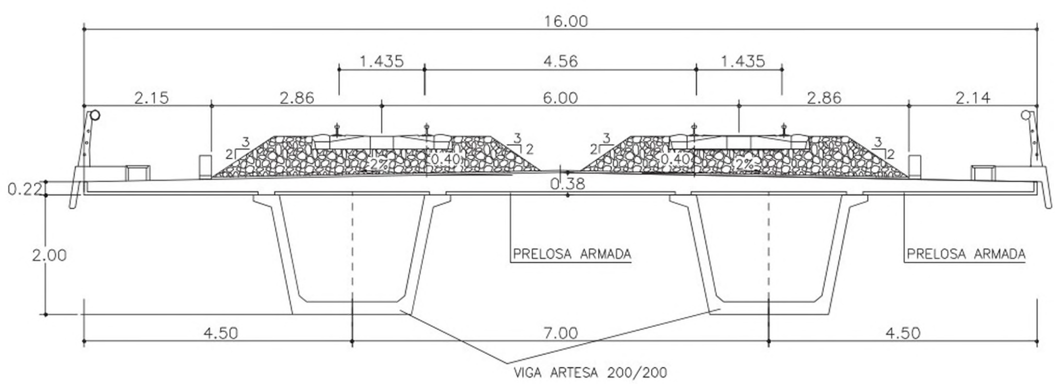

Figure 8. Typical cross section of Hontoria viaduct. HS Line Madrid–Valladolid. $L_{max} = 26.90$ m.

4 SPECIAL BRIDGES

For prestressed concrete girder bridges with spans of over 70 m, it is recommendable to use variable depth. The cross section remains that of a single-cell box with a depth at supports on the order of one L/13 and L/25 and in the midspan section. This type of bridge, then, is normally built by the balanced cantilever construction, with the segments poured on-site, using moving form travellers. Figure 13 shows the bridge over the Guadalquivir River, with a span of 80 m, for the Cordoba–Malaga high-speed line, designed by Carlos Alonso Cobo.

For longer spans, and taken as examples of what are now considered emblematic bridges, the French have used bownstring schemes (La Garde Adhemary and Mornas bridges[5]), both built of steel. The Germans have accomplished the same task with frames and arches with upper decks (Fig. 14). In Spain, the Madrid–Barcelona line crosses the Ebro River on a metallic arch with a lower deck, with a span of 125 m [6], and Javier Manterola has used a scheme of u-shaped cross section with lateral Vierendel beams of prestressed concrete, 9.15 m depth, to cross a central span of 120 m [7].

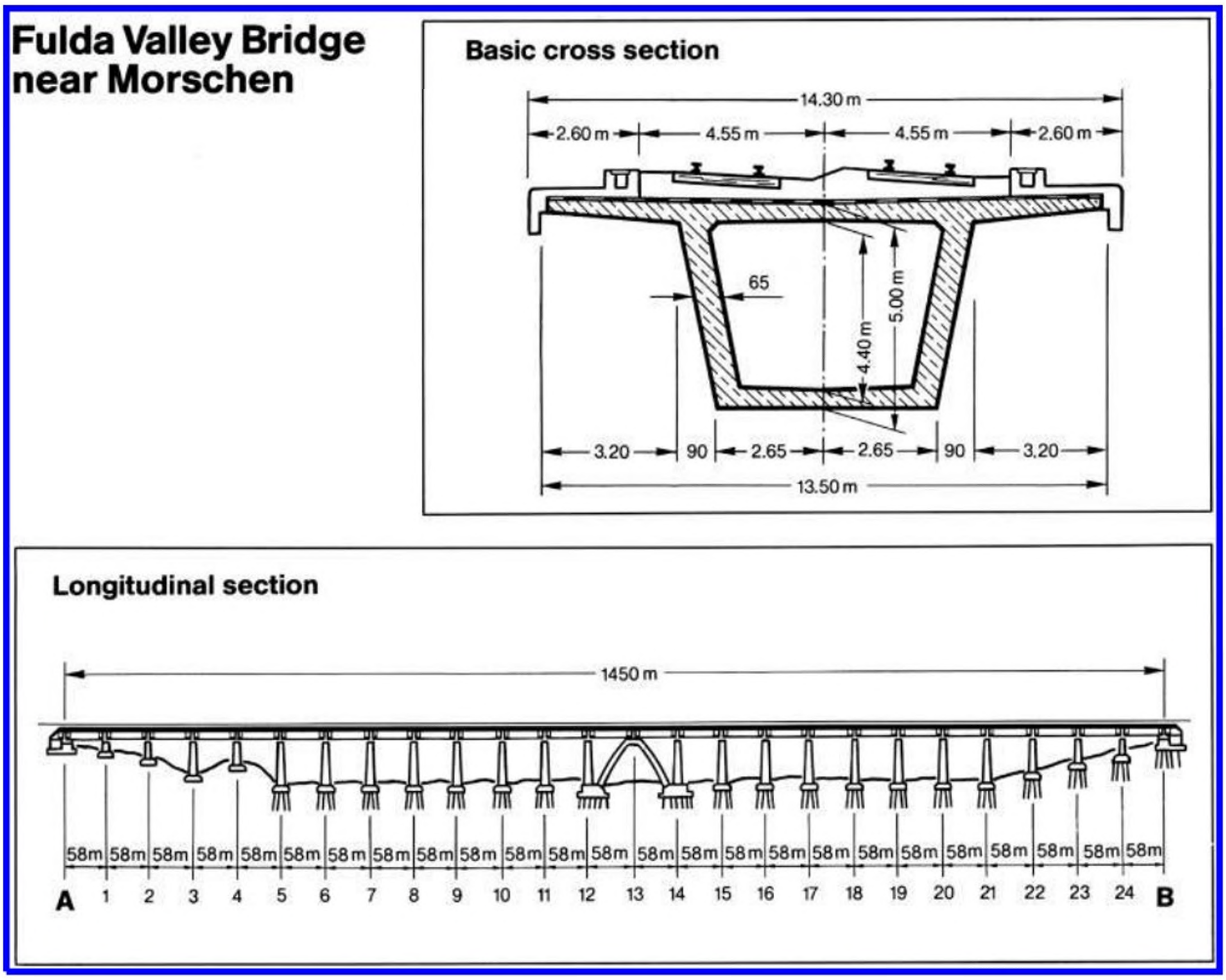

Figure 9. Longitudinal and cross section of Fulda viaduct. German HS Lines.

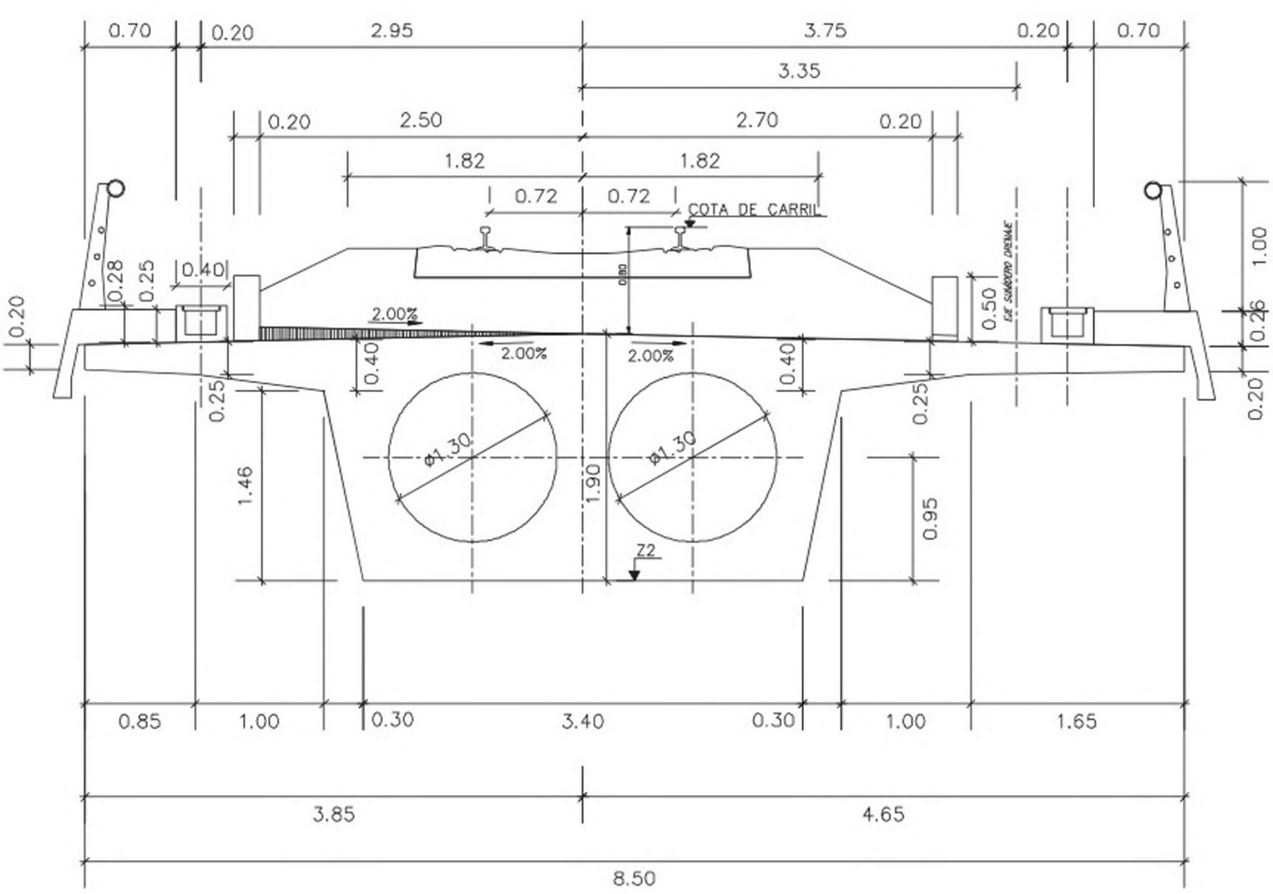

Figure 10. Typical cross section of Mas Borras viaduct. HS Line Lleida Barcelona $L_{max} = 30\,m$ (two separated decks).

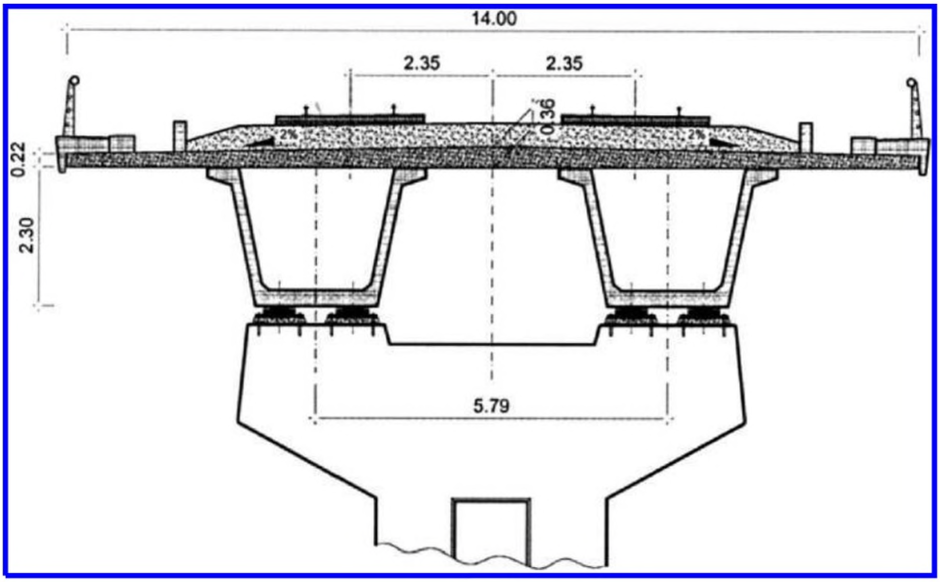

Figure 11. Martorella viaduct. HS Line Madrid–Barcelona. Two separate precast "U"-beams made continuous $L_{max} = 38$ m.

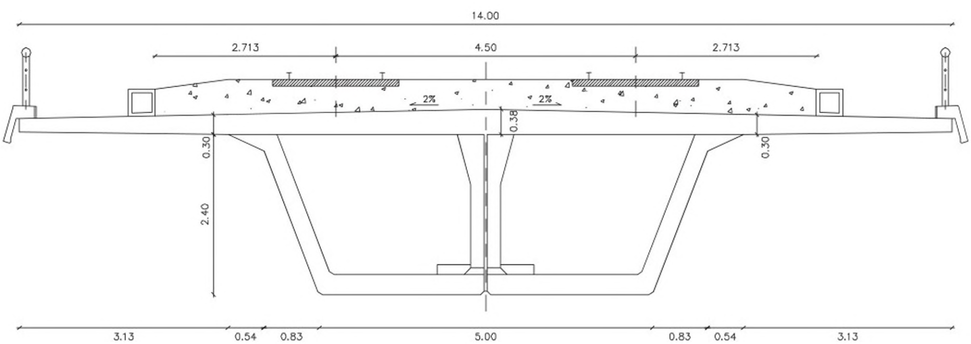

Figure 12. Ricla viaduct HS Line Madrid–Zaragoza. Two jointed precast "U"-beams made continuous $L_{max} = 38$ m.

5 PIERS AND ABUTMENTS

Where the scheme for supporting horizontal forces is that of a fixed point with the rest of the bearings slipping longitudinally, it is normal to use one of the abutments as the fixed point. That abutment, then, must be particularly strong, since it must be able to withstand very high forces and the maximum deformations caused by braking must be limited to 30 mm, due to the performances of the rail expansion joints. In addition, this abutment is almost always used as access to the interior of the deck for inspection, when the deck has a box cross section, and it must be possible to inspect also the devices used for anchoring the deck to the abutment, which we will discuss in the section on bearings devices. In respect of the abutment with moving bearings, there is no difference in comparison with highway bridges, except that there is no transition slab. It is precisely the transition by the rolling-stock from a flexible structure, namely an embankment, to a rigid structure, namely a deck, that constitutes one of the problems that has yet to be satisfactorily resolved in high-speed rail

Figure 13. Bridge over the Guadalquivir river HS Line Cordoba–Malaga. $L_{max} = 80$ m.

Figure 14. Bridge over Maine river. German HS Lines. $L_{max} = 162$ m.

lines. The use of wedges of cemented ground and cemented gravel has not been totally successful, and the foundation of such wedges and a part of the embankment by means of jet grouting is now being studied. On the Cordoba– Malaga line, the rail expansion joints are being placed on reinforced concrete structures founded to the embankment itself, to avoid problems of relative movements of the rail expansion joints.

Where the valley allows, a workable solution is the one used by the Germans for the Fulda viaduct, (Figs. 9 and 15). This viaduct, with typical spans of 58 m, is supported on conventional piers. When it reaches the riverbed, the corresponding pier is doubled to create an inverted "V", very suitable for resisting horizontal forces. This bridge also uses viscous shock absorbers on the abutments, which allow slow displacements while at the same time support instantaneous bearing braking forces.

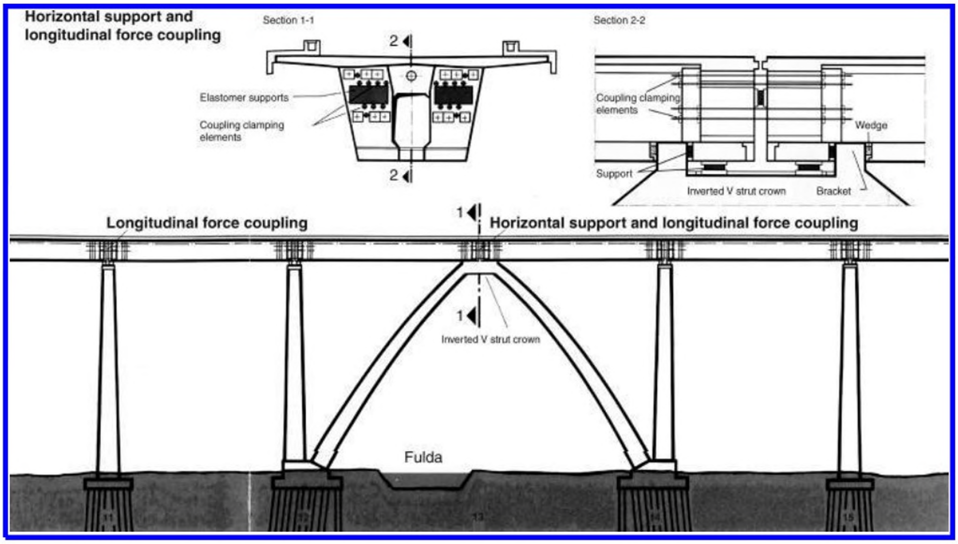

Figure 15. Longitudinal section of Fulda viaduct.

In respect of piers, in the case of resistant horizontal schemes with a fixed point on the abutment, in longitudinal direction, and with the bridge in service, they are only subjected to friction due to bearings devices. This friction can control the buckling length when the buckling occurs in the opposite sense. Nevertheless, in bridges built using the incremental launching method, the horizontal forces caused by friction during construction are of the same order of magnitude, but with a pure cantilever scheme, meaning that second-order effects in displacements and bending moments must be controlled. Crosswise, the piers are subjected at their heads to wind effects and centrifugal forces in curved bridges, and even in the case of large curvatures radius, speeds are also high and therefore important, since they give rise not only to internal forces but also to horizontal displacements that must be controlled in order to preserve the geometry of the track. In spite of this, piers can be relatively slender in areas with no seismic activity. In areas with seismic activity, the effects of such activity are critical, and we are of the opinion that the dissipation of energy through sectional ductility in a pure cantilever scheme is questionable in terms of equilibrium, and we would take a cautious approach. Nevertheless, a second-order analysis and a proper layout of reinforcement can save many amount of steel reinforcement.

6 SCHEMES RESISTANT TO HORIZONTAL ACTIONS AND BEARING DEVICES

The general criteria for preservation of the geometry of the track and for rail expansion joints required for high-speed railway are the following:

i) Longitudinal displacements must be limited to the following values:
   – instantaneous, $Ux <= 25$ mm (because of the characteristic of rail expansion joints)
   – long term, $Ux <= 300$ mm (to prevent disturbance of ballast)
ii) Crosswise movement of the end supports of the deck must be prevented (preservation of the geometry of the track)
iii) Least possible number of rail joints (comfort)
iv) In ramp or graded bridges, all bearing devices must be placed horizontally, except those for mobile abutment, which must be placed parallel to the slope to prevent vertical movement

v) In curved bridges, the retainer for unidirectional bearings must be placed tangentially to the plane axis and not in the direction of the radius vector with the fixed support. In this way, temperature and retraction produce forces, but discontinuities in the rail are avoided.

These criteria lead, owing to vertical loads and movements, to the exclusive use of bearing of the "POT" type made of confined neoprene or spherical bearing devices, with no use of the conventional reinforced neoprene bearing devices that are systematically used in highway bridges.

The major problem in railway bridges, in which they differ totally from highway bridges, is the magnitude of the horizontal forces. In fact, the fixed point of a railway viaduct 500 m in length, similar to the one in Figure 1b, must be able to withstand approximately 13400 kN of horizontal force, in both directions. Of course, the retainers of the "pot" bearing cannot withstand such extreme forces. This problem is solved by the use of additional bearings, usually made of reinforced neoprene, placed perpendicularly to the force. Since they must work in both directions, there are several possible solutions. Two usual solutions are: design a mortise in the fixed stirrup in the shape of a "c" on the plane and with four neoprene bearings, so that they withstand the horizontal force two by two (Elephants viaduct [8]), or prestress the deck against the abutment by means of prestressing tendons located along the centre line of the bridge, so that when the bearing are subjected to tension by the external force they decompress, and when they are compressed, this force is added to the prestressing. This is how the functions are specified: the bearings placed perpendicularly to the centre line of the deck withstand the horizontal forces, with their distortion allowing the rotation of the deck, while the "POT" bearings, moving freely in the longitudinal direction, withstand the vertical forces. In any case, it is important to bear in mind that in all viaducts over a certain length, the axial force caused by braking and linear deformations must be taken into account in verification of the SLS and ULS.

7 CONTROL OF FORCES IN RAILS AND LAYOUT OF RAIL EXPANSION JOINTS

Current railway technology uses 2000 m welded rails and this approach, based on the least possible number of joints between rails, is maintained in structures.

Furthermore, the temperature difference in winter and summer between rails and concrete or composite decks, owing to their different thermal inertia and the protection of the ballast, can be as great as $\pm 30°$ C. Therefore, over the active length of the rail, when the rail moves more than the deck or support, it generates on the deck or support forces of interaction with the deck or support that are calculated at ± 5 kN/m and rail, up to a maximum of 500 kN per row of rails (active length of the rail of 100 m). Depending on the placement of the rail expansion joints, this force will or will not be added to the force that the bearing need to withstand.

The transmission of horizontal forces between the rail and the structure occurs through the support of the ties on the ballast and has load-displacement behaviour that is linear up to 60 kN on a loaded rail and 20 kN on an unloaded rail. Above those values, the rail slips.

The presence or absence of track expansion joints at the ends of the structure can determine the presence or absence of forces of track-structure interaction, which in turn influence two phenomena: additional forces on the bearing and overstresses in the rails that must be limited to $\Delta\sigma \leq 72$ MPa in compression, to avoid lateral buckling, and to $\Delta\sigma \leq 92$ MPa in tension.

In theory, and with the aim of ensuring comfort, the number of rail expansion joints (REJs) must be kept as low as possible. So, that stretches with total length of under 60 m in steel bridges or 90 m for composite and concrete ones will not need REJs at their ends. In this way, the braking forces acting on the deck are distributed among the fixed bearing of the structure, and the rest of the continuous rail.

Where the static scheme is a succession of simply supported spans, the best approach is to link them in respect of horizontal actions to a fixed point, lay continuous rail over all the spans and support each span on moving bearings. In this way, the horizontal forces on the rails are transmitted to the structure and the structure, being continuous, carries them to the fixed point. REJs are placed

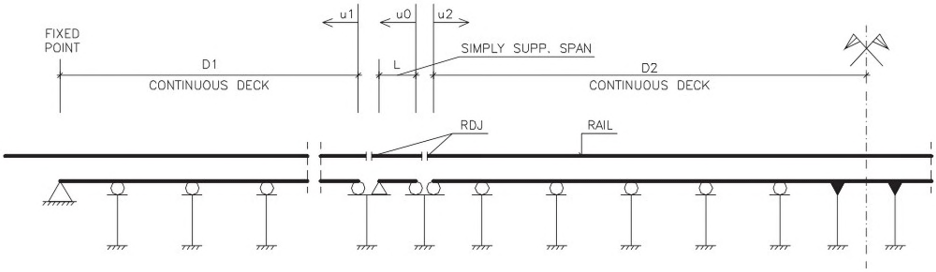

Figure 16. Rail expansion joints layout for a very long viaduct.

at the fixed point and at the opposite end. The linkage should be made by means of the centred prestressing of one span against another using neoprene plaques also placed on the centre line of the deck, Figure 15 rather than by means of continuity slabs, as are used for highway bridges. Such slabs should be forbidden for railway bridges, since they do not withstand fatigue.

The other two possible solutions are not advisable: if we lay continuous rail without linking the decks, the braking force is distributed among the piers and rails, and if the piers are not sufficiently rigid, the rails are overloaded and can buckle. On the other hand, if we place REJs at either end of each span, the number of REJs makes smooth rolling impossible.

In continuous viaducts of considerable length, the best approach is to place an REJ on the moving abutment and lay continuous rail over the whole structure and the fixed abutment. In this case there are interaction forces through the structure that are added to the bearing friction and braking forces and must be considered on the fixed point. We have already mentioned that the maximum deferred movement that can be tolerated without disturbing the ballast at the juncture of the structure and the bulkhead of the moving abutment is 300 mm, which limits the continuous length of the deck to one that insures that the deformations occurring after the track is laid are less than that amount. Since when that occurs a substantial portion of the shortenings due to shrinkage and creep in concrete bridges have already taken place, lengths of between 450 and 600 m are possible. To attain greater lengths in viaducts, the solution is to place the fixed point at the centre, either with a pier in the shape of an inverted "V", as mentioned earlier, or to introduce a short span, with a frame scheme, with the deck fixed into both piers, and to place both moving abutments, with or without shock absorbers, in such a way as to limit braking movements. In exceptional cases, such as the TGV viaduct near Lyon, both schemes can be used Figure 16 continuous stretch with the left abutment fixed – inert simply supported span – continuous central stretch with intermediate frame and symmetrical.

8 CONSTRUCTION METHODS

In the field of short spans, both on-site construction on formwork and erection of prefabricated beams using cranes are used. In the area of medium-length spans, on-site construction on formwork is preferred for short, low viaducts, since, where the height of the viaduct is greater, modern cranes allow easy placement of prefabricated beams, although with the substantial environmental cost involved in the opening of access roads to the bottom of the valley for installation of the cranes. For long viaducts, the preferred methods are span by span construction using mobile scaffoldings and incremental launching method. The advantage offered by the span-by-span method is its total adaptability to any plane layout, while the advantage of the incremental launching method is found in its economical cost and speed, in spite of the disadvantage of lesser adaptability in respect of layout, although there are ways of adapting it (layout of the bottom of the box on a circular plan with variable cantilevers, construction with a web of variable height, construction on the basis of circular vertical agreement and descent of supports, etc. [9]). Construction by means of the balanced cantilever method is useful for spans of over 70 m where there is a lower obstacle to be spanned, or various spans that justify the cost of the form travellers. Bridges built with prefabricated segments have been widely used for light rapid transit in the United States and France, but not for viaducts on high-speed rail lines. Otherwise, construction methods for special bridges with long spans apply the standards used in construction of highway bridges.

9 CONCLUSIONS

The intensity of vertical and horizontal loads, their frequency and dynamic character, comfort parameters required an the track geometric maintenance are conditions for the design of HSR bridges.

This paper deals with the above mentioned requirements and gives specific design criteria for such bridges with the aim that an experience engineer in designing highway bridges could easily adapt his expertise to HSR bridge projects.

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CHAPTER 16

The cable-stayed bridge over the Po river

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ABSTRACT: The new High Speed railway linking Bologna to Milano crosses the Po with an 11 Km long viaduct, the most important of the new line. The distance between the main embankments is 1.2 Km, the ordinary riverbed being 350 m wide.

Three types of structure are present in this part: (i) the cable stayed bridge, whose central span is 192 m long (ii) 12 simply supported 45 m decks and (iii) two continuous p.c. box girders overpassing the main embankments.

The cable-stayed deck is a p.c. continuous box girder; the towers are 60 m high and have foundations supported by 28 piles, 2 m diameter and 65 m long, each.

Special studies for the vibrations and the track-structure interactions were necessary, the operating and design speed being respectively 300 and 350 km/h. Furthermore many tests have been carried out to validate the numerical models.

The construction of the bridge started at 2002 and lasted about 4 years.

1 INTRODUCTION

The new High Speed railway linking Bologna to Milano crosses the Po near Piacenza in a section where the river is usually about 350 m wide, up to 1 km between the main embankments. The bridge is 1200 m long, 400 m to cross the ordinary riverbed, an obliquity of 22° resulting between the tracks and the river. Two approach viaducts, respectively 6 and 4 km long, complete this work, the most important of the Bologna Milano line. A number of restraints have guided the design, the most outstanding being (i) the navigability; (ii) the scour; (iii) the environmental impact and (iv) the seismicity of the zone.

A clear span of 70 m was requested for the navigability, that resulted in a minimum distance between the piers of about 90 m, taking into account the obliquity. Four main spans of 96 m were proposed in the preliminary design to satisfy this requirement, and two solutions were selected after a first study (Fig. 1), but the competent Authority for the Environment insisted in eliminating the central pier, a 192m main span resulting for that fact.

The reduction of the number of piers in the riverbed was a choice confirmed by the elevated scour expected, up to 16 m, that increases the cost of the foundations. The low seismicity of the zone must be also emphasized: a PGA of 0.15 g being to be expected within a return period of 500 years, the local risk analysis said.

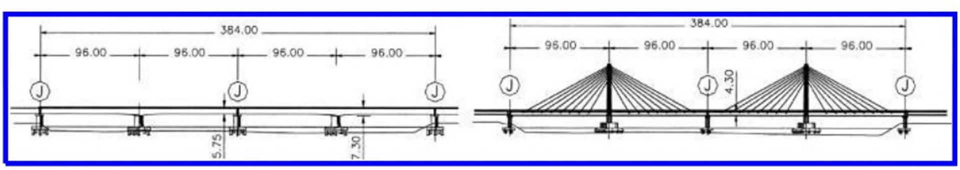


Figure 1. Preliminary proposed solutions.

2 BRIDGE'S MAIN FEATURES

2.1 *The general layout*

Three types of structure are present in the crossing in addition to the standard approach viaducts placed outside the upper banks: the cable stayed bridge, 12 simply supported decks on the right bank and two continuous p.c. box girders necessary to overpass the main embankments (Fig. 2).

The decks are subdivided in such a way that two joints in the rails are necessary to keep the expansion length within the allowable limits. This will be the only exception along the jointless Italian HS railway network.

2.2 *The overpasses of the embankments and the simply supported spans*

An Italian law forbids any type of construction at a distance less than 10 m from the foot of the embankments. This restraint, together with the high obliquity, meant large spans also to overpass the main embankments.

Two continuous p.c. box girder have been designed for that: five spans of 37-67.69-51.4 × 3 on the left side and three spans of 33.4-62.7-33.4 on the right side. Its cross section is very similar to that of the 13 simply supported spans on the right bank (Figs. 3a, b).

2.3 *The cable-stayed bridge*

The relevant part of the crossing has a 192 m central span and two 104 m long side spans (Fig. 4). The deck is a p.c. continuous box girder with the fixed point at one tower, sliding bearings at the

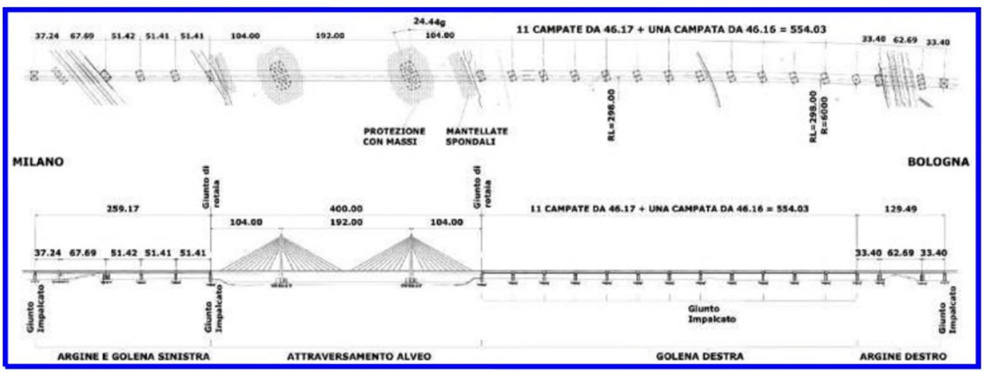
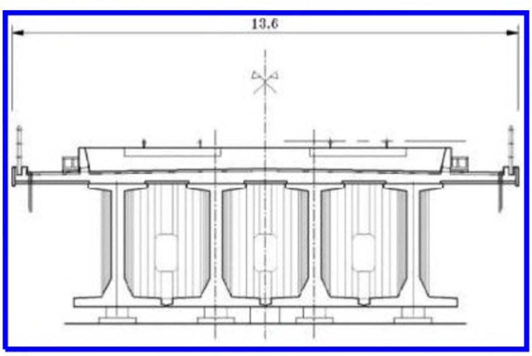


Figure 2. General views of the whole bridge.



(a)

(b)

Figure 3(a–b). Approach spans cross section and the actual element under construction.

second tower and at the transition piers. Expansion lengths of 296 and 104 m derived from this arrangement of the bearings, so requiring joints in the rails. The height of the cross section is constant and equal to 4.5 m (L/42.7) along the central span; it varies and decreases to 3,70 m in the side spans, in order to join the other decks.

Prestressed transversal diaphragms are placed where the stays are anchored to the deck. In this zones also the lateral webs are prestressed vertically by bars. The total width is 15.7 m, 2.1 m larger than the standard sections: that allows to keep the planes of the stays outside the electrical supports, with a large allowance with respect to the nearest rail.

The towers are 60 m high from the footing, 51 m from the deck (Figs. 5a, b). The need to have the base of the tower orientated like the stream and the upper part orthogonal to the rails, i.e. rotated by 22°, originated the singular shape of this element.

The top of the towers, where the stays are anchored, is a steel-concrete composite structure. The inside steel box supports the horizontal components of the forces in the stays, while the concrete placed outside is stressed by the vertical components. This solution, very frequent in the new bridges, avoided having too many rebars and is easy to built.

The stays are made of zinc-coated, singularly greased and sheeted 0.6″ strands, whose number for each stay varies from 55 to 91. The overall sheet of each stay is made by light-grey HDPE initially supposed to be grouted by mortar. The tests described hereafter suggested avoiding the grouting according to the recent tendency in this field.

The total amount of steel for the stays is 410 tons, corresponding to about 66 Kg per square meter of deck.

The foundation of each tower has the footing (shaped to reduce the drag force) supported by 28 piles, 2 m diameter and 65 m long.

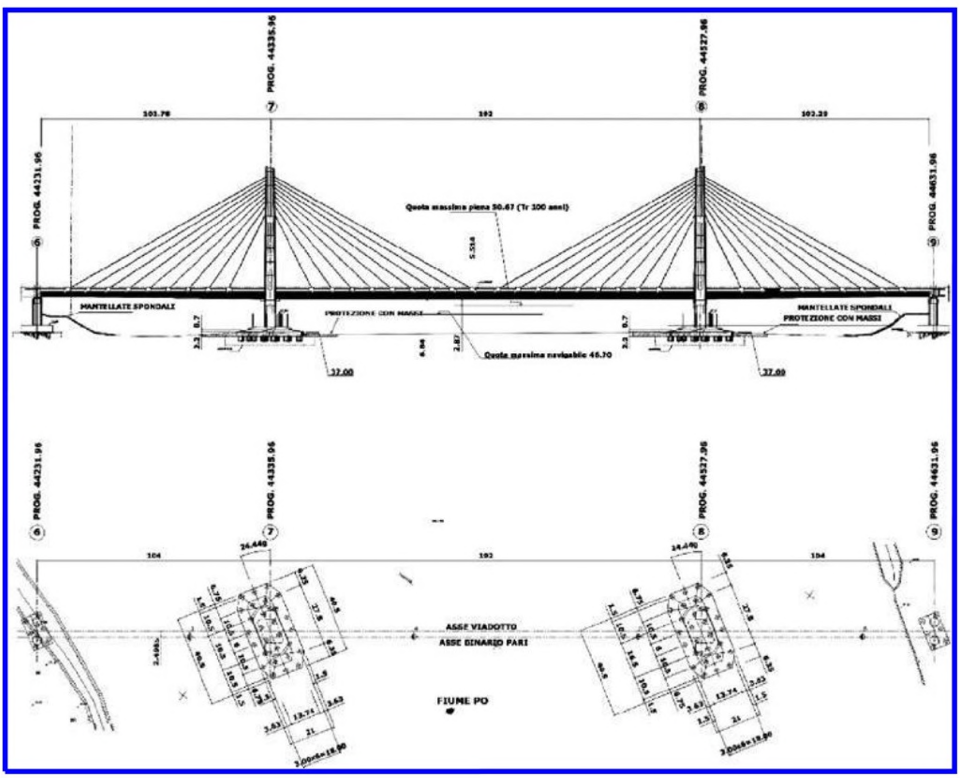
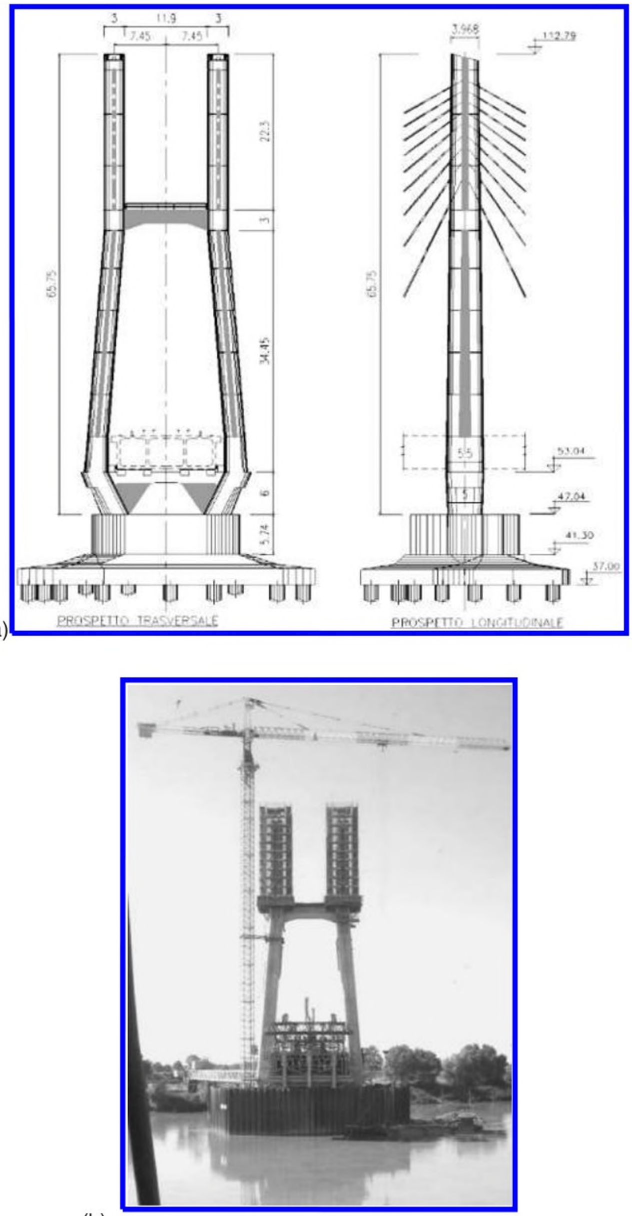


Figure 4. General views of the cable supported deck.



(a)

(b)

Figure 5(a–b). Front and side views of the tower and the same element under construction.

3 DESIGN CRITERIA

Besides the usual design criteria prescribed by Ferrovie dello Stato-ASA Servizi di Ingegneria (1997) and by prEN 1991-2 for bridges equipped by conventional ballasted track, a number of additional assumptions were required.

3.1 *Segmental construction of the deck*

A fixed amount of rebars must pass through the junctions; in these sections, moreover, a residual compression stress of 0.5 MPa must be assured under the most severe service loads (1 MPa without the thermal effects).

3.2 *Derailment of railway vehicles*

Two design situations have been considered with respect to the stays:

– collapse of two consecutive stays along one side due to the derailment of a vehicle: the bridge must remain in service with one design train over the track nearest to the injured side and one passenger train (40 KN/m) over the other, the thermal effects being excluded;
– the consecutive stays collapsed are three: only the effects due to one design train is taken into account.

3.3 *Scour*

Two design situations have been considered also in this case. For the maximum expected scour of 16 m the seismic actions have been excluded and only the stress for a single design train have been checked: the displacements of the structure, and therefore the passenger comfort, have been ignored. A scour of 8 m has been considered "frequent" and all the Service Limit States have been satisfied.

3.4 *Dynamic analysis*

Three different trains (ETR 500, TGV, ICE) have been considered for the dynamic analysis that has been undertaken in three steps: (i) a simplified model considering only moving forces has allowed in a preliminary stage to select the most critical cases with speed ranging from 150 to 360 km/h; (ii) a complete global analysis considering the dynamic behaviour of the vehicle as well as the irregularities of the track allowed to check the dynamic impacts (iii) local dynamic interaction in the zones near the towers and the rail joints has finally showed the safety factors for the derailment and the overturning, these analysis having been performed only for ETR 500 and for two speeds: 280 and 360 Km/h (1.2 time the operating speed).

The maximum impact factor for the bending moment at midspan resulted to be 1.64 while the maximum amplification of the deflection was 1.45. The peak vertical acceleration of the coach, for the speed of 360 km/h, resulted to be about 1 m/s^2 and was due mainly to the track irregularities. The maximum derailment and overturning coefficients with open joint and lateral wind were also within the allowable limits.

3.5 *Combined response of structure and track*

The effects resulting from variable actions have been taken into account according to prEN 1991-2 and for two limit stiffness of the foundations. Figure 6 shows the rail absolute maximum stresses (above) and the relative longitudinal displacement between the track and the deck (below), both far below the allowable limits.

Seismic analysis have been carried out in the elastic range according to the European Code 8, with an elastic modulus of each stay linearized around the permanent value. Fifty vibration modes have been considered, the first one being clearly due to the vertical vibration of the deck while the second is related to its longitudinal movement (Fig. 7).

The mass of two trains weighting respectively 80 and 40 KN/m have been added to the permanent loads as requested by the Italian Railway (Ferrovie dello Stato 1996), while a probabilistic risk analysis showed that no scour had to be considered during earthquakes.

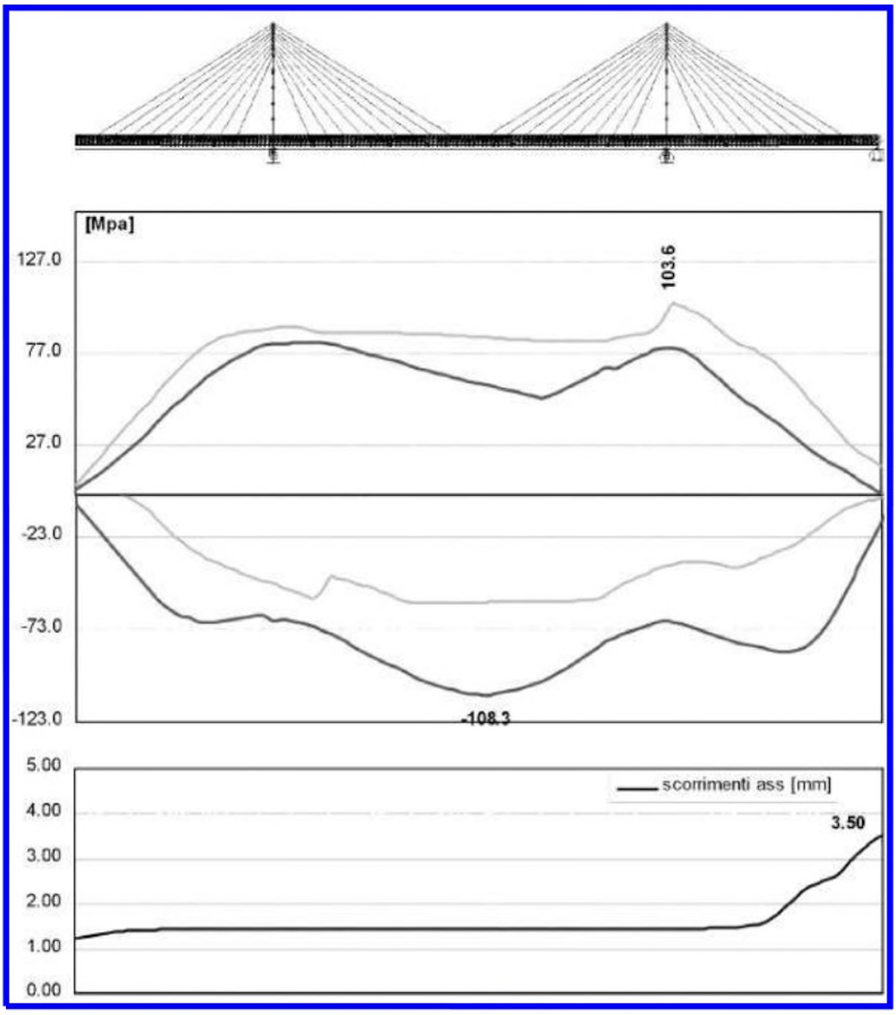


Figure 6. Maximum rail stresses and relative displacements.

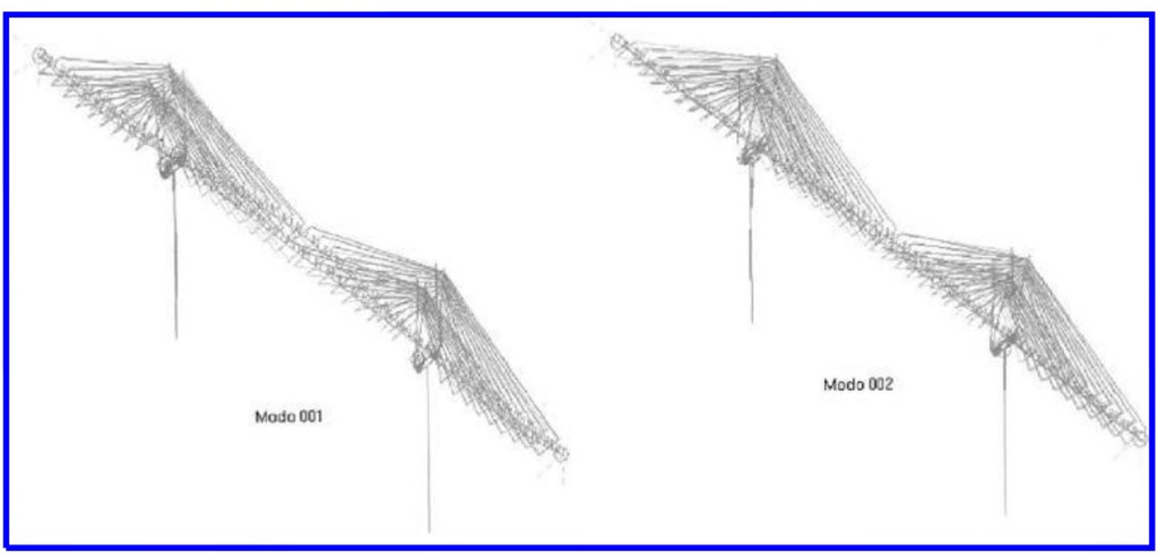


Figure 7. The first two natural mode of vibration.

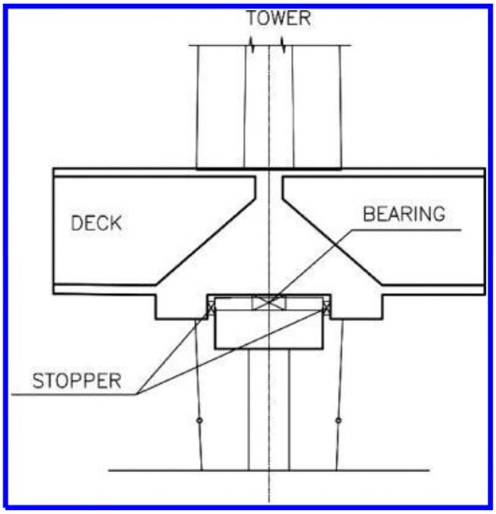


Figure 8. Deck section with view of the bearing devices.

Figure 9. View of the main foundations under construction.

Because of the low seismicity, seismic actions did not influence the design of the bridge but in a few sections in the upper part of the towers, while they were relevant for the bearings and the joints. The bearing devices at the fixed point have been specialized, appropriate stoppers having been charged to support the relevant horizontal forces (Fig. 8).

4 CONSTRUCTION METHOD

4.1 *Foundations of the towers*

The steel cases of the piles, and immediately after the steel sheets of the cofferdam, have been driven in a preliminary stage in the riverbed operating by pontoon.

Two artificial islands have then been built, a provisional deck made by steel beams and concrete movable slabs having been placed on the top of the cofferdam (Fig. 9).

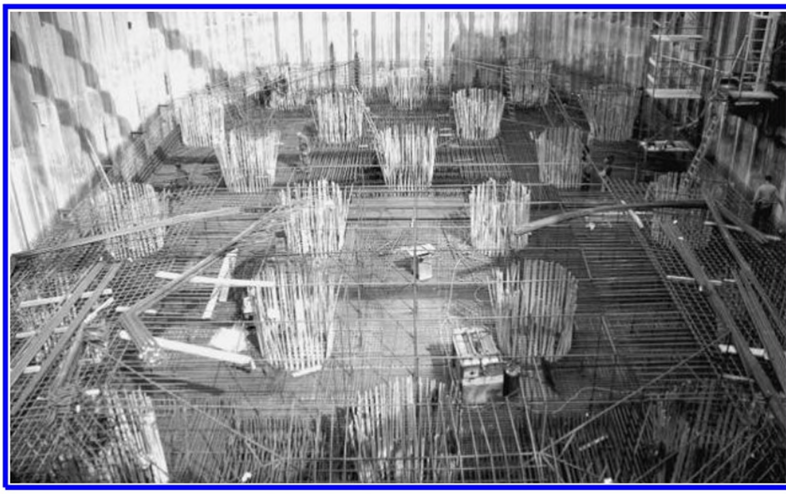


Figure 10. View of the foundations after pumping of water.

(a) (b)

Figure 11(a–b). Segmental construction of the approach spans and cable-stayed bridge decks.

Heavy boring machines, standing over these decks, built the 2 m diameter 65 m long concrete piles, with water still inside the cofferdam. After that a 4 m thick concrete slab has been poured at the bottom of each foundation and the water has been pumped away (Fig. 10). The buoyancy was counteracted by the concrete slab and by the steel cases of the piles adequately anchored to it.

4.2 *Decks*

All the decks were built by cantilever method with cast in situ segments (Figs. 11a, b), but the 12 simply supported spans. These were made by four precast T beams jointed together and prestressed transversally in order to have a box cross section. Each segment of the cable-stayed deck was 2600 KN heavy and 4.50 m long so that one of two brings the stays anchorage. The typical sequence of construction was:

– cast in situ of a segment with anchorage and advancing of the self supporting formwork;
– first tensioning of the correspondent stay;

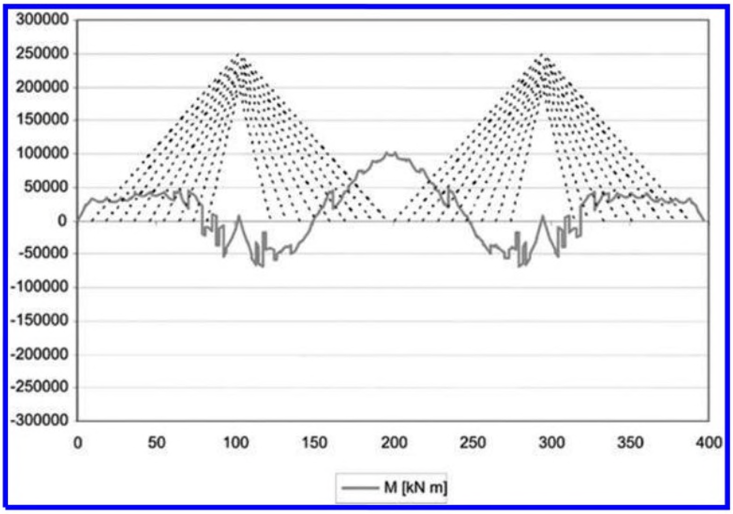


Figure 12. Permanent bending moment distribution on the cable-stayed bridge deck.

– advancing of the formwork and cast of the next segment without stay;
– second tensioning of the stay.

A final regulation of all the stays has been carried on once completed the full deck, so that each stay was stressed 3 times in total.

At the end of the construction the bending moment distribution in the deck was as shown in Figure 12 to partially counteract the service stress due to the ballast and to the trains. So, only very few prestressing cables were necessary, about 40 Kg/m^2, to guarantee the residual compression requested under the most severe conditions.

5 MODELS AND TESTS

A number of physical tests were executed to assess the theoretical assumptions. These were:

(a) Tests by Osterberg cell on two piles expressly built, 2 m diameter and respectively 50 and 55 m long. A load of up to 20.8 MN (1.37 times the maximum service load) has been reached. The measured settlements were in a good agreement with the expected ones, differences less than 10% resulting from the tests;

(b) Test on a half scale model reproducing a segment of the deck with a stay anchorage (Petrangeli & Polastri 2004). This test was carried on in the yard by the Dpt of Structural Eng. of the University of Rome (Figs. 13a, b, 14a, b) with the aim to investigate the stresses distribution near the anchorage. About 400 strain-gages have been utilized and a load equivalent to 1.5 the maximum service force in the stay has been applied. The strain gauges have been placed on this model, appropriately distributed on the rebars, inside the concrete and on its surface. A preliminary check of the whole control system and of the stability of the instruments has been performed by reading them every 15 minutes with no load. The instruments tolerance has been fixed at $+/-35\,\mu\varepsilon$; a strain-gauge was considered reliable if a drift smaller than the tolerance was measured during a period of time similar to those expected for each phase of the test. So doing only 325 channels, i.e. about 80% of the total, has been considered stable (Petrangeli & Cipolloni 2005).

After that, the reliability of a strain-gauge during the test has been assessed if the measured value was within two limits defined by the theoretical value ε_{ex} derived by FE model plus or minus a fixed deviation Δ, with:

$$\Delta = (0.2\varepsilon_{ex} + \varepsilon_0)$$

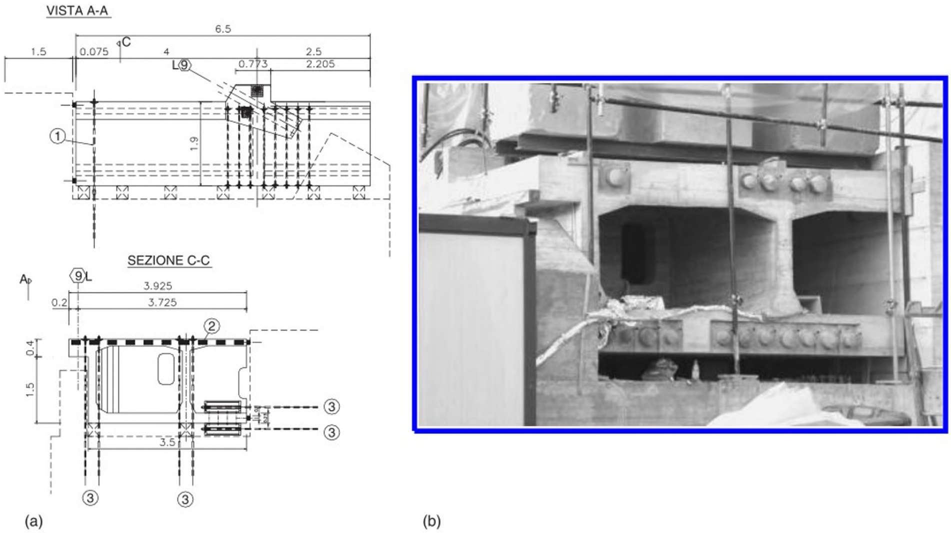


Figure 13. Half scale model of deck segment sections and actual element.

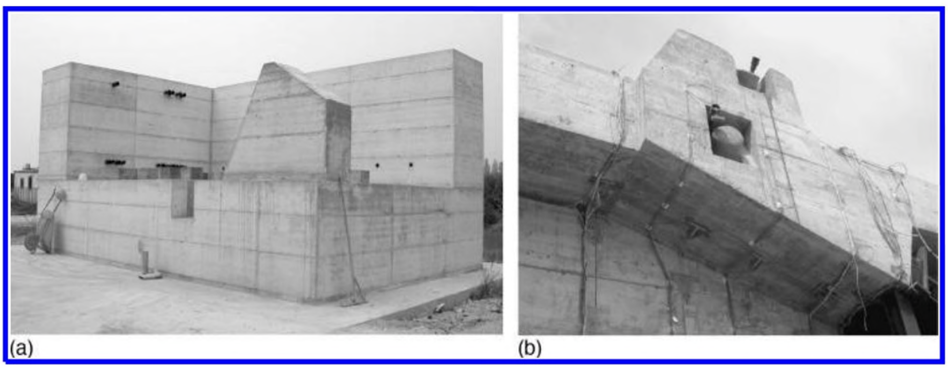


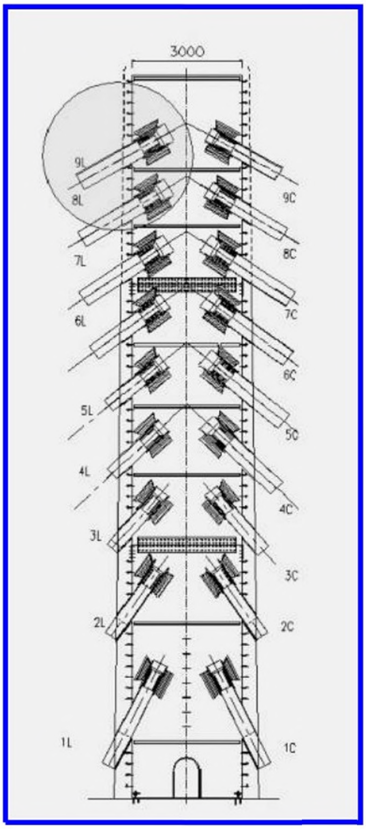
Figure 14. The basement before the placing of the model and detail of the anchorage.

ε_0 is the already mentioned instrumental tolerance of 35 $\mu\varepsilon$.

The measured stress confirmed the theoretical analysis very satisfactorily as far as the force in the stay was less than the expected one; over this limit some local non linearity appeared but no cracks have been experienced for the maximum test load.

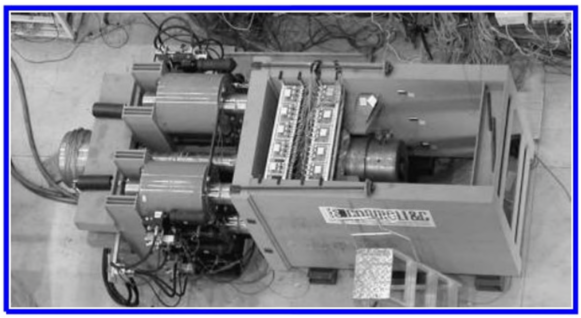
(c) Fatigue test on a full scale model of the steel box embedded in the upper part of the tower to anchor the stays (Petrangeli et al. 2004). This test was carried on in the Structural Division (ELSA) of the Joint Research Centre (JRC) of the European Community. A force varying between 6700 and 7900 KN for two millions of cycles was applied to the model shown in Figure 15a, b, c. Also in this case a good agreement between the physical and mathematical models was found (Figs. 16a, b). A force up to 1.3 times the in service expected maximum was reached (the limit was due to the testing equipment) without any visible damage in the tools and in the welded connections.

(d) Fatigue tests on three stays (composed by 55, 73 and 91 0.6″ strands) complete of anchorages. Also these two million cycle tests were carried out in the ELSA Laboratory according to a procedure derived by the US Post Tensioning Institute (PTI) rules (Post-Tensioning Institute 2001). It was the first time a fatigue test on a 91 strands cable has been undertaken in Europe.



(a)

(b)



(c)

Figure 15(a–c). Section of the top of the tower and two images of the full scale model of an anchorage.

One of the three samples was stressed simultaneously by the main longitudinal force and by a secondary transversal force simulating the wind effects (Fig. 17).

The test on the 55 strands stay was carried on a model including all the non structural parts like wax in the anchorage, sheat and so on. At the time grouting by mortar was foreseen and therefore it was present in the model but, according to the results obtained, it was decided to eliminate it because it could damage the protection of the single strand.

In this case, therefore, the test was meaningful not only to check the strength of the stay, but also to validate the construction procedure and the reliability of the protections against the corrosion.

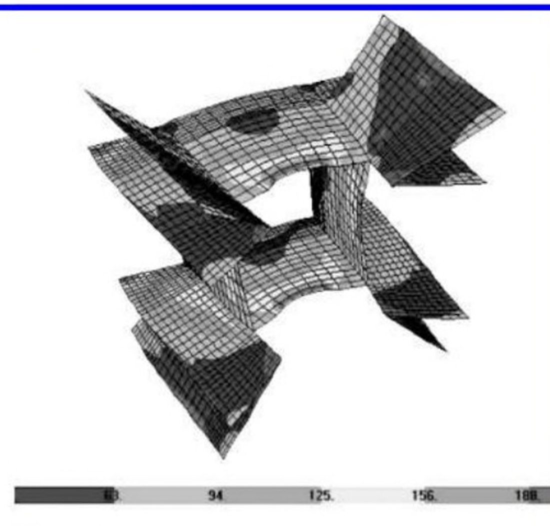
(e) 1:50 model test to assess the scour (Fig. 18). Those tests have been carried on in the PROTECNO SrL Laboratory, Padova; the expected shape of the scour was confirmed while the entity of the scour resulted 20% less severe than the theoretical.

6 PERMANENT MONITORING

Due to the importance of the bridge, a large number of sensors have been placed on it. The monitored quantities are: loads on the piles, stress and temperature in the most representative sections of the deck and the towers as well, forces transmitted by a number of stays and bearings, geometrical data

	tensioni attese	tensioni lette	
[KN] N=	9200	letture	
Codice estensimetro	σ_{id} [MPa]	σ_{id} [MPa]	$\Delta\sigma_{id}$ [%]
O1 b4	130	155	19.36
O1 b1	160	90	-43.76
O1 d3	239	144	-40.02
O1 f1	85	100	17.84
O2 a1	234	116	-50.61
O2 f1	100	69	-30.78
O4 e1	130	55	-57.79
O4 a1	180	92	-49.03
O4 d3	239	182	-23.84
O4 d4	239	170	-29.09
O4 b4	120	147	22.52
O3 f1	90	94	4.12
O3 d2	140	120	-13.96
O3 c2	160	38	-76.17
O3 a1	230	109	-52.49
V2 dB	210	67	-68.26
V2 dC	200	107	-46.37
V2 dD	210	71	-66.29
O2d4	239	185	-22.58
O3d4	239	199	-16.95

(a)



(b)

Figure 16(a,b). Expected and measured stress on the anchorage model.

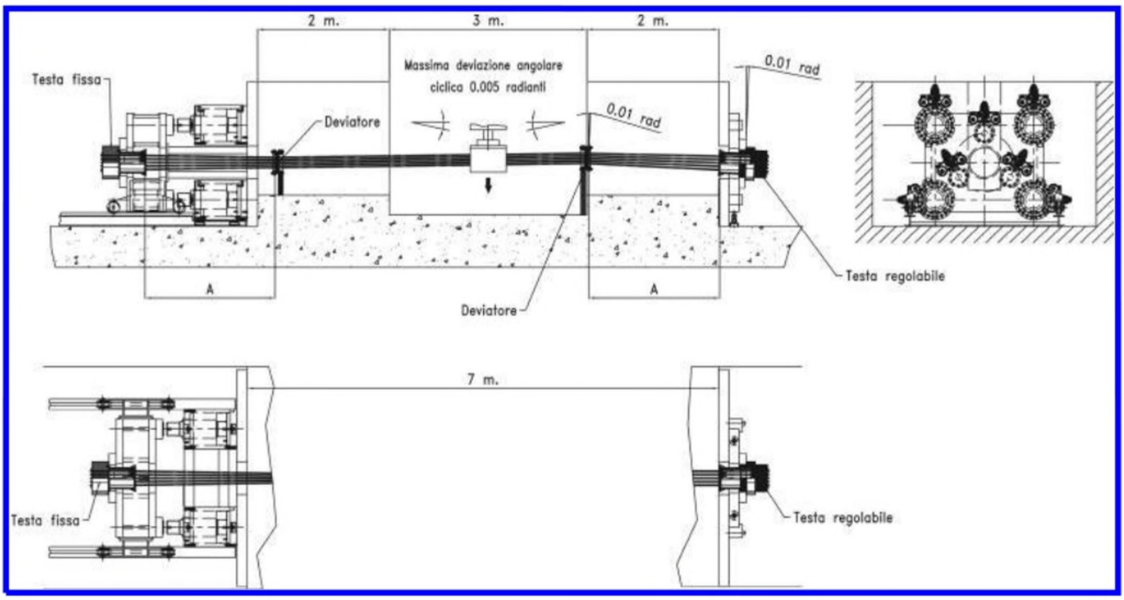


Figure 17. Set up of the test with transversal force.

like the angular rotation of towers and the deflection of the decks and, finally, the scour near the piers in the riverbed.

Both sonar and magnetic devices will be installed to detect scour.

All the data are collected inside the cable-stayed deck and from there automatically transmitted to a remote office that will manage all the bridges of the line. The cost of all the equipment was about 1.5% of the total cost of the bridge.

All these instruments have been used since the construction phases. The first results stressed the temperature play an essential role in the understanding of the bridge behaviour during these phases. Large difference have been recorded not only between the stays and the deck as well as the upper and lower face of the box section, as usual, but also between different segments. That was

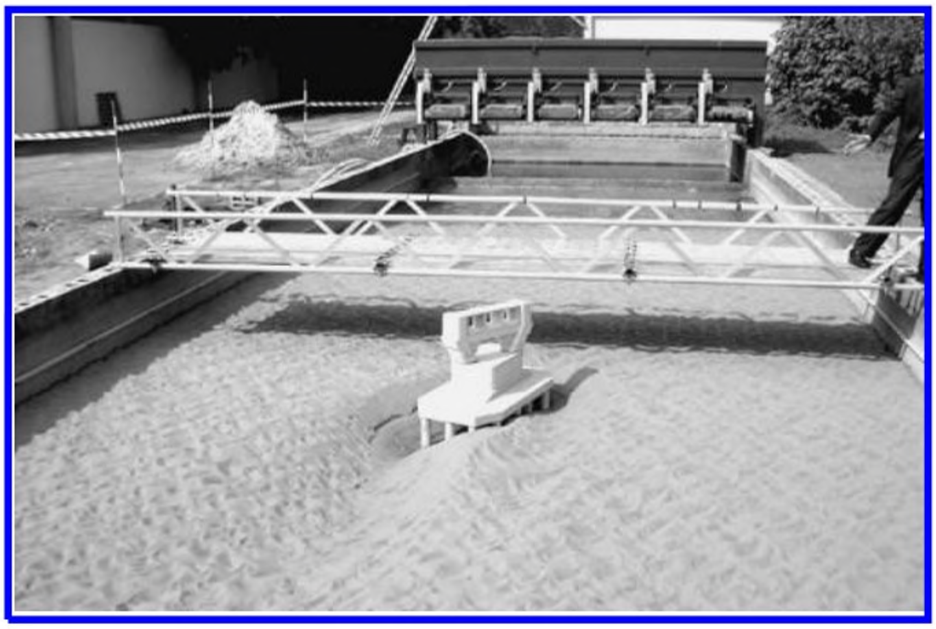


Figure 18. Hydraulic 1:50 model to investigate the scour.

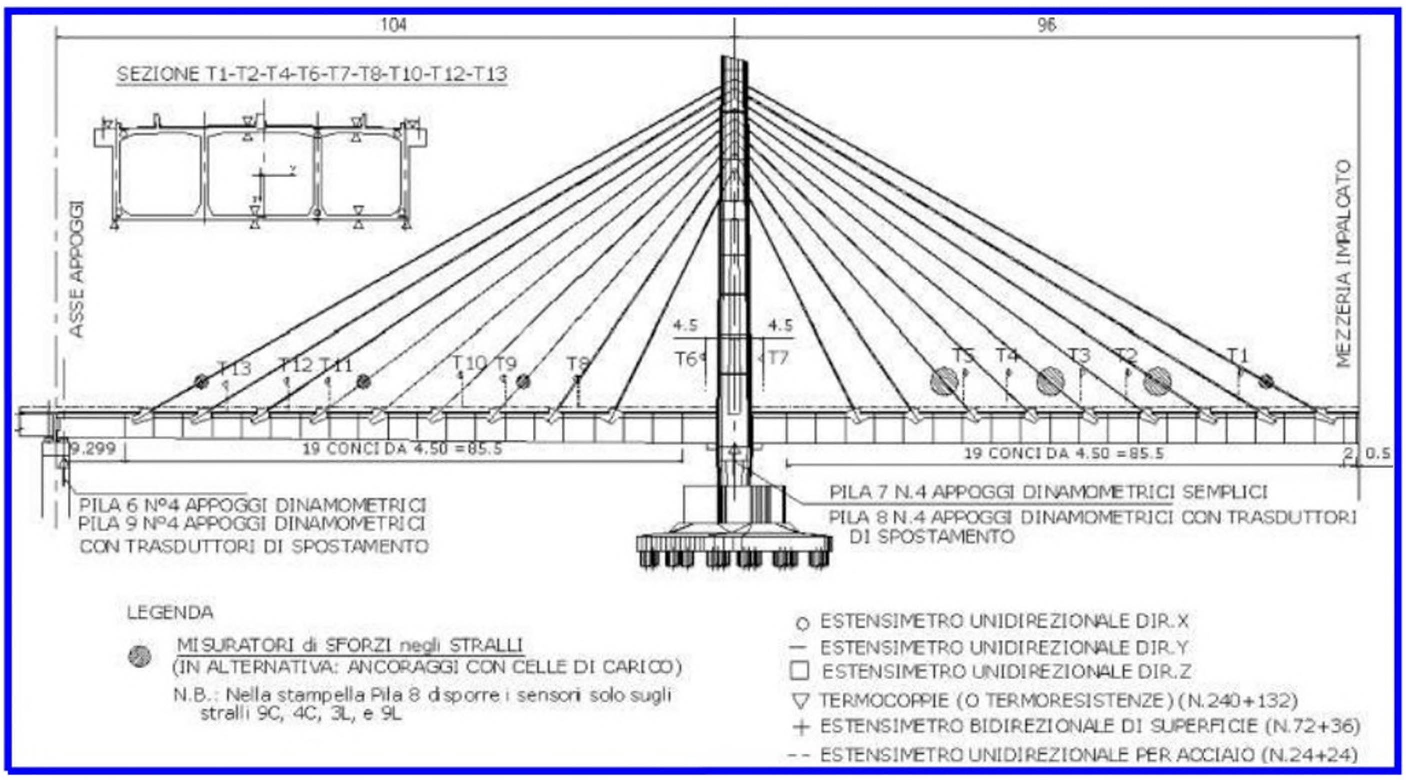


Figure 19. General layout of the permanent monitoring.

probably due to the different circulation of the air inside the box according to the distance from the free cantilever section.

7 CONCLUSIONS

The bridge over the Po river confirmed the stays are a economical and aesthetical solution also for large structures built for High Speed railway (Fig. 20). The stiffness requested to these bridges to guarantee the passenger comfort, also for speed as high as 350 km/h, can be reached with a relatively slender deck also in the cases where prestressed concrete is used.

Cable stayed bridges result therefore as a valid solution alternative to the arch bridges or to the steel trusses.

Figure 20. The cable stayed bridge in winter 2006 before removing the cofferdams and the scaffolding at the top of the towers.

ACKNOWLEDGEMENTS

TAV S.p.A. is the Company fully shared by RFI, Italian National Railway, established to built and to manage all the High Speed Italian network.

ITALFERR S.p.A., also shared by RFI, was the Supervisor for the design and construction of the bridge by appointment of RFI.

CEPAV1, a consortium headed by ENI, was the main Contractor for the whole line Bologna – Milano.

ASG Scarl (AQUATER, SNAMPROGETTI and GRANDI LAVORI-FINCOSIT) was in charge of the construction by appointment of CEPAV1.

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CHAPTER 17

Composite and prestressed concrete solutions for very long viaducts: analysis of different structural designs for the Spanish high speed lines

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ABSTRACT: A wide programme for development of high speed railway infrastructure is being carried out in Spain. This paper presents the most common structural forms for long viaducts which are being designed in our country. Special emphasis is drawn on the selection of the structural scheme in order to successfully solve the resistance requirements for longitudinal starting-off and braking forces and the displacement tolerances of the railway expansion joints. The most common prestressed concrete solutions are presented, and the different construction methods are detailed: prefabrication, advancing shoring and launching. The steel concrete composite alternatives for the deck are also presented in comparison with concrete bridges. These alternatives have been widely used in Europe but they are only starting to be a design option for railway bridges in Spain.

1 INTRODUCTION

The development of high speed railway infrastructure is now underway in Spain. Two high speed lines are operating yet. The first of them links Madrid with Seville. It was opened in 1992 to coincide with the Expo '92. The second one has recently been opened between Madrid and Catalonia. These lines are the first examples of a wide planning of railway infrastructure connecting the main cities of Spain by high speed railway lines. This programme demands a huge amount of investment which has been guaranteed for at least the duration of the present decade.

The physical geography of our country give rise to a large number of structures in order to maintain performance requirements of high speed lines. These structures are frequently several hundred meters long, but in the last years the design of bridges longer than a kilometre has become usual. For example, the bridge over river Jalón extends 2268 meters in the high speed line between Madrid and Barcelona, and it is not an exception.

The I beam simple span solution was employed in several bridges along the high speed line between Madrid and Seville. However, the greater dynamic and serviceability conditions derived by the high design speeds of up to 350 km/h on the new lines have practically discarded this solution, mainly as result of warping. This type of construction tends to be limited to pergola solutions or structures of low importance.

Continuous prestressed multi-span deck is the basic solution currently adopted in most of the bridges along the new high speed railway lines. A series of simply supported spans are usually employed in long viaducts with low pier heights and good foundation conditions. When necessary, continuous multi-span decks are located into the series of simply supported spans to solve some specific circumstances, i.e. a greater span locally required. Of course, continuous decks are useful when the height of the piers, span length or foundation conditions requires it.

These decks tend to be formed by a single cast in place box cross section erected by launching or by self-bearing bracing systems. For spans less than 35 meters, a voided slab have also been designed and built span by span on scaffolding, and precast solutions have also been successfully employed with spans of up to 45 meters when crane assembly was allowed by ground conditions. Opened precast prestressed box cross section with upper slab cast in place to close the section was used in these cases, both in simply supported or continuous beam over supports.

Concrete solutions were always chosen in Spain for the first high speed lines. In fact, all bridges in both lines operating are concrete bridges. Steel concrete composite decks have good acceptance for road bridges, but they were not considered in the beginning by the entities responsible for the design and construction of railway high speed infrastructure in our country. However, an exhaustive study was carried out by the authors of this paper about possible steel concrete composite solutions for the bridge over Arroyo Las Piedras, in the high speed line between Córdoba and Málaga, [4]. Since the very beginning in the design, we realized that a better response to the specific conditions of the bridge would be provided by a steel concrete composite solution for the deck, considering the height of the piers, the length of the bridge, seismic requirements and foundation conditions. For the first time in our country, the proposal for a steel concrete composite deck for a high speed railway bridge was accepted by the authorities, and it has been finally constructed. References [5][6][7][8] describe the adopted solution in detail.

2 LONG BRIDGES IN HIGH SPEED LINES: LONGITUDINAL FORCES AND EXPANSION JOINTS REQUIREMENTS

Strict slope and plan requirements imposed by the high speed railway lines at present make it frequently necessary the erection of great structures to solve properly both the specific environmental and functional conditions. Many long bridges are then required, with hundred of meters between abutments or even more, and height of the piers also important.

The adoption of the longitudinal restraints of the static and dynamic model constitutes the main problem in the design of these bridges. These bridges introduce a special singularity in the platform. Therefore, they must be designed with specific conditions to provide an adequate behaviour under the traffic loads which generally determines the structural morphology. The basic aspects to be considered are summarized in the following:

a) Location of rigid points to resist horizontal actions applied, according to foundation conditions, height of the piers and building procedure. Braking and start off forces from traffic loads must be withdrawn with minimum movements. These forces are really important for long bridges. They usually reach the higher values established by Regulations, 6000 kN/track from braking force and 1000 kN/track from start off force. Braking in one track must be simultaneously considered with start off in the second one for the frequent case of lines with two tracks.

   As we have mentioned before, these forces must be withdrawn by the substructure with minimum movements in the deck. The maximum horizontal displacement of the deck is limited to ± 5 mm if the rail has no expansion devices at one or both of the abutments, and ± 30 mm if rail expansion devices are located at both abutments.
b) Analysis of the track-structure interaction which exists under rheological and thermal actions and braking forces, so as to maintain the rail in adequate conditions. Stresses in the rail must be properly controlled, and relative displacements between track and structure must be limited as well [12]. Finally, total displacement accumulated at the expansion device is also important. As a result, location of fixed points and expansion devices along the bridge constitutes a critical step of the structural design. Some solutions have specifically been developed to solve these questions, for example the classical intermediate simply supported span located into a continuous beam to limit the total displacement accumulated at the abutment. Fixed points are then located at both abutments and expansion devices appear at both ends of the intermediate simply supported span. The total displacement is then half that with a fixed point at one abutment and expansion device at the opposite one.

The distance between expansion devices in the bridge is the main parameter in the analysis. As the length of the bridges is tending to increase progressively as a result of environmental stipulations, the study of the track-structure interaction has turned critical. Up to ten years ago, total displacement in rail expansion devices were generally limited to 450 mm. However, recent technical developments allows the current regulations of ADIF, the entity responsible for the construction and maintenance of high speed lines in Spain, to limit the total displacement to 1200 mm.

The movement of track expansion devices is a consequence of the rheological and thermal displacements of the deck. We can estimate these average movements in 1.20 mm/m for prestressed concrete bridges and 0.70 mm/m for steel concrete composite solutions. It means that the maximum expansion length in the bridges may vary between 800 and 1000 meters applying the current regulations of ADIF. The incidence of rheological creep and shrinkage movements is very sensitive to the actual sequence between the end of construction and the welding of the continuous rail and the installation of the track expansion systems. However, these aspects may be considered in the specific design of the expansion devices, but they are generally unknown in the design of the bridge.

In spite of all these improvements, track expansion systems continue to be a weak point for the safety, comfort and maintenance of the track. The behaviour of the balast in the platform is affected by the relative displacements between track and structure, and high stress levels may be developed in the rails, so these aspects must be studied in detail in the project.

The basic solution to solve the specific requirements above mentioned is one of the following:

I. A series of simply supported spans where braking and start off forces in each span are withdrawn by one of their piers. As we have mentioned before, the total displacement of the deck under traffic loads is drastically limited, so piers with high stiffness must be designed. As a result, this solution applies only when piers do not exceed 15/20 meters in height, with good foundation conditions.

   Longitudinal displacements from thermal and rheological behaviour of the deck are not accumulated at the abutments. Therefore, the track-structure interaction is reduced. A low level of stresses is introduced in the rails because of the relative movement between the end of the decks and the upper rail, but this is very low if the stiffness of the piers reach the adequate value. So, no track expansion devices are generally necessary at the ends of the bridge.

   This structural form has successfully been used in our country with bridges longer than 2000 meters.

II. A continuous deck with fixed point at one abutment and expansion device at the opposite one. This is an excellent alternative when the high of the piers or foundation conditions do not allow the transmission of horizontal forces span by span as the previous, and they are accumulated to the abutment.

   The fixed point is generally located at the abutment which shows better foundation conditions, and the lowest height if it is also possible. Longitudinal forces are usually withdrawn by friction between the ground and the abutment, so this is frequently a massive element to provide the weight necessary.

   An expansion track device is located at one abutment to allow the total movement accumulated. As a result, the maximum length of the bridge is limited to 800/1000 meters, according to the expansion devices requirements above mentioned.

III. A continuous deck with fixed points at both abutments and location of an intermediate simply supported span in the central zone of the deck to allow the movements of the lateral continuous decks at both sides, each one transferring longitudinal forces to its abutment. This is a great alternative when the total length of the bridge is over the maximum for the alternative II) and the height of the piers or foundation conditions do not allow the alternative I).

   The simply supported central span may also be a series of few continuous spans, but always separated from the lateral decks at both sides. As a result, the longitudinal movement of each side may be treated separately, and the track expansion devices require a total movement lower than that from the total length of the bridge. Expansion devices are located at both ends of the intermediate isolated spans, with continuous rails at both abutments.

An adequate response to braking and start off forces acting on the isolated spans must be guaranteed in the design. The stiffness of piers is then the main parameter to be considered. Sometimes it is necessary to increase the stiffness of these piers from that of the rest of the bridge. Foundation conditions may also require specific studies.

The above mentioned solutions are the most frequent alternatives in the design of long high speed railway bridges. However, some specific conditions could turn them impossible and other alternatives must be developed. In these cases, the adoption of structural forms allowing for the horizontal forces without relevant displacements must always be considered in the design: arches, bow-strings, frames with inverted V piers, or rigid piers V or delta shaped, as shown in reference [12].

Longitudinal impact transfer devices at abutments may also be successfully employed when specific conditions do not allowed a conventional alternative. If seismic requirements are relevant, it constitutes an excellent solution when combines damper function in the devices. Low speed movements are allowed without resistance, but horizontal serviceable forces are achieved without relevant displacements. In addition, transferred force to substructure is limited when an earthquake takes place [5].

3 PRESTRESSED CONCRETE VIADUCTS

The basic ideas which have been mentioned before are the guide for an appropriate selection of the structural form of the bridge. However, a combination of several forms is also possible. For example, the design of simply supported spans for longitudinal bending which are connected between them with short prestressing for longitudinal forces and imposed deformations. This structural form has been widely used in Germany, where a rapid and easy replacement of the deck if necessary is required by the authorities. But this may also be an interesting alternative for simply supported spans with self bearing falsework construction when the longitudinal stiffness of substructure is not guaranteed, because of the height of the piers or foundation conditions.

Simply supported spans are generally designed with depth/span ratio in the range between 1/11 and 1/15. They are easy for cast in place construction with self bearing falsework equipments, but precast elements erected by cranes are also frequently used.

Continuous solutions are designed with depth/span ratio between 1/14 and 1/18. They allowed for a launching erection or self bearing falsework equipments. Continuous bridges in Spain have also been made of precast elements erected span by span with cranes. Once the beams are located in the bridge they are connected between them with prestressing at hogging cross sections to make a continuous beam. Conventional prestressing systems at the upper slab cast in place have been used, but short prestressing with bars at the U shaped precast elements have also been frequently used. Both have been successfully designed with spans up to 40 meters long when the erection with cranes of precast elements is allowed by the height of the piers and ground conditions.

Both for precast or cast in place construction the optimum in the length span is usually about 30 meters to 45 meters, according to technical and economical conditions from the height of the piers and foundation requirements. Larger spans in the range up to 65 meters are also possible, but they are only designed when substructure conditions are severe. Of course, a larger span is designed if a river or a natural obstacle in the line requires it.

Some of the most relevant bridges we have designed in our office for railway high speed lines in Spain are described in the following.

3.1 *Bridge over river Cinca*

The bridge over river Cinca was the first long bridge in the High Speed Line between Madrid and Barcelona, [11] and [10]. It is 830 meters long with a series of 14 spans 58 meters long, and three spans 70 meters long each one over the main bed of the river. The depth of the deck is 4.85 meters.

Figure 1. Bridge over river Cinca.

Figure 2. Launching of bridge over river Cinca.

Classicist lines were particularly taken into account in the design of the structure. Horizontal character of the deck was remarked, supported by a series of octogonal shaped prismatic columns as piers. Concrete was painted in white (Fig. 1).

Just one fixed point was designed at one abutment, with a track expansion device at the opposite. Self weight is over 300000 kN and it all was launched from one abutment. For the first time in Spain, launching of this range of tons was made from one abutment (Fig. 2).

3.2 *Bridge over river Ginel*

The bridge over river Ginel is located near the town of Zaragoza, in the high speed line between Madrid and Barcelona. It is 1228 meters long and the average height of the piers is 43 meters. Fixed points were designed at both abutments, where two lateral zones of the deck were anchored with 648 meters and 536 meters respectively. A simply supported span 44 meters long is located between the main lateral zones in order to limit the movement of expansion joints. This form was described in the previous section of the paper (Fig. 3).

The depth of the deck is 4.0 meters, with a span 48 meters long. The depth/span ratio is 1/12. This a low value but it was successfully used with external prestressing to simplify reinforcement steel. As a result, an easy launching procedure was obtained (Fig. 4).

Figure 3. Bridge over river Ginel.

Figure 4. External prestressing.

The launching was made from both abutments. The simply supported central span was launched provisionally connected to the lateral zone during launching.

The economical success of this building procedure lays on a good conception of falsework equipments and reinforcement steel details which allow for a semi-industrial process in the launching area. At least three launching operations each two weeks are required (Fig. 5).

3.3 *Bridge over river Jalón*

This is the longest bridge in the high speed line between Madrid and Barcelona. It is 2.238 meters long with a series of 56 simply supported spans 34.50 meters long each one, and two intermediate continuous zones 165 meters and 156 meters long with spans up to 48 meters over the main bed of the river and the highway N-II respectively. The depth is 3 meters all along the bridge, with a depth/span ratio 1/11.5 clearly appropriate for this configuration (Fig. 6).

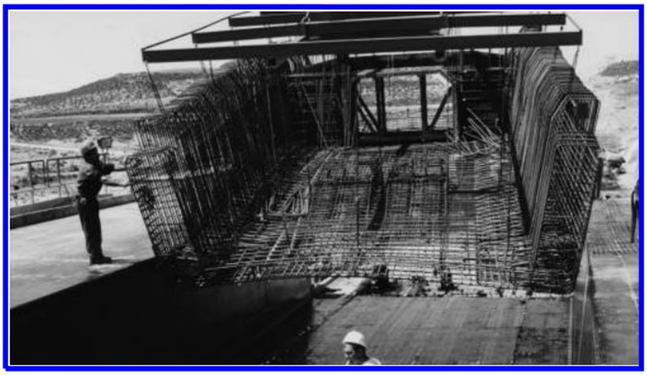

Figure 5. Semi-industrial process in the launching area.

Figure 6. Bridge over river Jalón.

Span by span construction with self bearing falsework equipments was designed (Fig. 7).

Most attention was dedicated to simplify details in the design, so as to reach the best sequence in the construction. Reinforcement steel and prestressing strands were manufactured out of the falsework equipments, and then the whole steel was introduced in the falsework. Two spans each three weeks were erected during construction (Fig. 8).

3.4 *Bridge over river Genil*

The bridge over river Genil is located at the high speed line between Córdoba and Málaga. Its construction has recently been over. The deck is made of a series of 29 spans 48 meters long each one and the depth is 3.80 meters. The height of the piers varies from 10 meters to 31.50 meters. As the low stiffness of the highest piers is not appropriate, a rigid pedestal at the bottom of the piers higher than 22 meters was designed to maintain the length of the column below this value. As a result, an adequate behaviour of the piers submitted to longitudinal horizontal actions is obtained.

3.5 *Precast concrete bridges for the high speed lines*

As we have mentioned before, precast solutions have been successfully employed in Spain for high speed railway bridges with spans from 25 meters up to 40 meters long. This range of span is usually the optimum for most of the railway bridges.

Figure 7. Self bearing falsework equipment.

Figure 8. Whole reinforcement steel manufactured.

Figure 9. View of bridge over river Genil.

Figure 10. Simply supported precast spans (35 meters long).

Figure 11. Continuous precast concrete viaduct.

Simply supported spans (Fig. 10) or continuous beams (Fig. 11) are possible, depending the length of the bridge, location of fixed points, height of the piers and foundation conditions. Continuous solutions allow for prestressing at the upper slab, but predominantly just reinforced solutions with short prestressing made of bars or straight short strands between precast elements are allowed as well. Fatigue, cracking and deformations are rigorously controlled in the design.

Cross section is generally made of two open cross sections prestressed precast elements with upper slab cast in place connecting the beams and closing the section. The total depth/span ratio (precast element + upper slab) generally varies between 1/11 and 1/14 (Fig. 12).

Some particular conditions may require specific solutions. The bridge over river Jarama shows singular elements with variable depth over piers to allow for a span of 60 meters. Connection with conventional prestressing system between precast elements is designed. The bridge is located near Madrid in the high speed line between Madrid and Barcelona (Fig. 13).

4 STEEL CONCRETE COMPOSITE VIADUCTS

The bridge over Arroyo "Las Piedras" is the first application of the steel concrete composite continuous deck in the high speed railway line in Spain. Construction of this bridge has been finished in 2006.

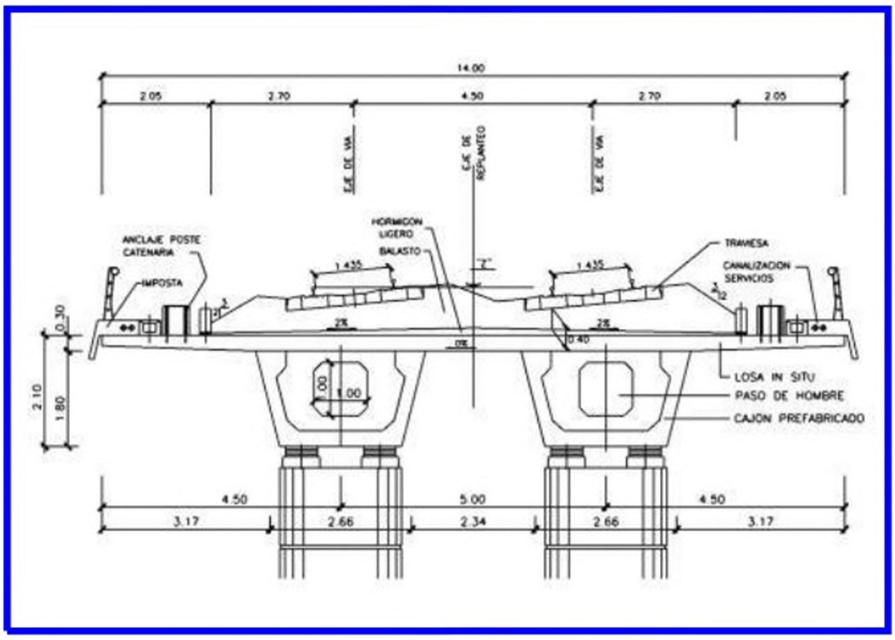

Figure 12. Cross section of precast deck with upper slab cast in place.

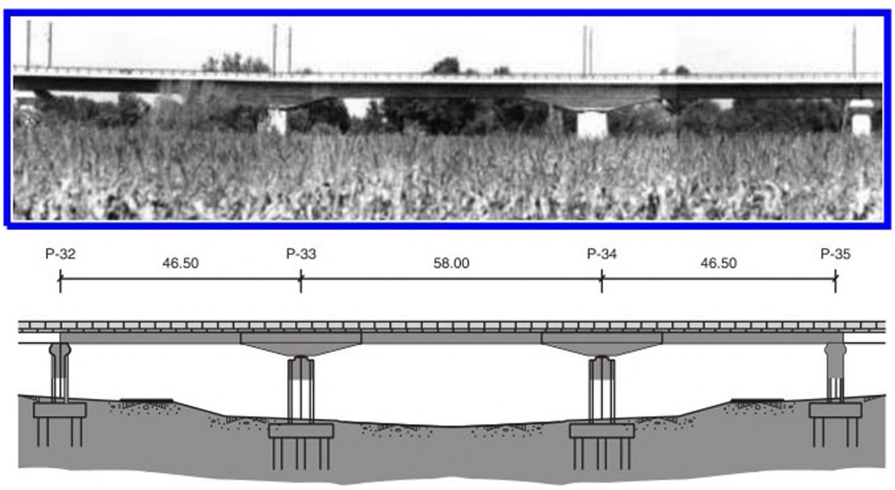

Figure 13. Bridge over river Jarama. Front view.

Figure 14. Bridge over river Jarama. Construction view.

Figure 15. Precast concrete girder.

The bridge is located at a highly seismic area. The total length of the bridge is about 1.220 meters between expansion joints, and it is made of a series of spans 63.5 meters long, a month the world's longest spans for this structural form of high speed line viaduct. Several piers of the bridge reach 92 meters in height. They have require detailed dynamic analysis and innovative structural solutions for resisting horizontal loads due too starting and braking forces, wind and seismic accelerations, which include the use of seismic isolation systems placed between the deck and the abutments.

The bridge is constructed using the incremental launching system. The steel-concrete composite strict box cross section with double composite action is adapted from the classic French twin-plate-girder structural form. It achieves several advantages such as a greater bending and torsional stiffnesses, significant reduction in the necessary reinforcement and structural steel ratios and a notorious construction simplicity.

The basic initial design of the bridge was a prestressed continuous deck with spans 50 meters long. The building procedure was segmental construction using self bearing falsework equipments. However, this initial design has finally been replaced by a steel concrete composite deck with spans 63.50 meters long, which was finally built.

4.1 *The bridge over Arroyo Las Piedras: Description of the initial concrete design*

The bridge over Arroyo Las Piedras is located at the high speed line between Córdoba and Málaga. It is 1.220 meters long. The piers height range from 10 to 92 meters. The initial design consists on a series of 23 continuous spans 50 meters long and two lateral counterbalancing spans 35 meters long. The cross-section of the deck (Fig. 16) is a conventional prestressed concrete box of a constant depth of 3 meters all along the structure length (slenderness of 1/16.66).

The 50 meters span sequence was a consequence of the maximum economic and resistant limits applicable in Spain to the segmental construction technique using self-bearing falsework. The conventional deck slenderness of 1/14 was slightly decreased in order to reduce the concrete weight on the falsework.

The self-bearing falsework system was agreed after a detailed comparative study with the launched alternative. Both solutions were approximately equivalent from an economic point of view. The launched system was rejected due to technical and economic reasons:

– The cost per m^2 was slightly higher than that of the self-bearing falsework system. The structure was to be launched from both abutments and a double launching park was required. Some technical problems, explained below, made it still more expensive.

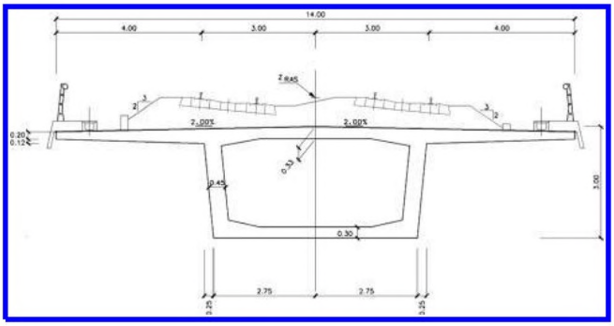

Figure 16. Mid-span cross-section. Concrete solution (50 meters span).

– The longitudinal plan and elevation could not to be modified due to the strict conditions of the general layout of the track. The solution was not apt for launching from a geometrical point of view. The corrective measures to make this possible were later adopted afterwards for the steel solution. However, those measures were more penalising for both the budget and the deadline of the works when applied to the concrete solution.
– The launching system for the concrete solution, of an approximate weight 20 t/m, was not regarded as technically advisable due to the great piers height of about 90 meters. There were considerable risks of excessive movements at the top of the piers. The piers were founded on long piles making it difficult to control loading on the piers shafts and the foundation bases. The possibility of developing great friction at the launching pads and the difficulty of recovering the vertical deflection at the top of the piers made it advisable to reject this system. The only possible solution consisted on allocating sophisticated launching control systems and eventually correcting deflections using jacks, which could make it difficult to evaluate the cost during the launching phase as well as delays in the construction stage of the bridge. Moreover, there was no other reference in the world to be extrapolated and taken as an example. The self-bearing falsework system appeared as a clearer and simpler solution.

The piers height and the environmental conditions to access the track ruled out any solution based on erection of precast girders.

The main problem of the viaduct project was the adoption of either longitudinal restraints or fixed points in the static model. This question is usual when total lengths are greater than 400/500 meters. The conditions were extreme in our case:

– The great length of the viaduct prevented from assuming just one fixed point at one abutment. The maximum range of the expansion/contraction total joint movements accumulated at the opposite abutment exceeded the maximum admissible limit of the rail expansion control joints used in the high-speed tracks.
– The great height of the piers and the bad foundation conditions made it technically and economically impossible to represent a fixed point in the middle of the viaduct in order to reduce the total movement of the expansion joints placed at both abutments.

For the first time in the history of the high-speed train in our country it was decided to design a solution to solve duality the fixed point/expansion joints structural response. The idea was to use impact damper-transfer devices at both abutments plus an expansion joint at a certain point in the deck.

The solution developed to modelize the longitudinal restraint in the structure consisted on:

– A virtual fixed point in the middle of the structure for long-term movements (shrinkage, creep, thermal movements) with free expansion joints at both abutments.

– A fixed point at each abutment for the braking force represented by an impact damper-transfer device. The central piers had longitudinal guided bearings that could undertake a small fraction of the breaking force. However, not all the forces could be transmited to the piles due to their enormous flexibility compared with the stiffness of the abutments.

These devices, which consist of a battery of hydraulic damper acted as fixed points against short-term forces. (breaking and traction forces in the case of an impact transfer) and variable stiffness springs (bilinear model) against seismic effects reducing the transferred force (seismic dissipation function). They are not a real restraint for long-term movements.

The piers of the viaducts were designed in a hollow section with variable depth. The cross-section was a rectangle of constant width transversally to the axis of the bridge and coincident with the base of the deck box. The dimension changes longitudinally ranging from 2.50 meters at the top and a maximum of 5.50 meters at the base of a 92 meters high pier.

The foundations of the piers were deep foundations. The pile caps were supported on 4 to 9 piles of 1.800/2.000 mm diameter and 25 to 35 meters long. The substrata consisted on soft brown clay and loamy clay layers over a resistant sandstone layer.

The viaduct is placed in a section of the track of a medium-high seismic intensity with basic accelerations of 0.08 g. The seismic force turned out to be dimensioning for the structural design.

The thermal isolation system designed for the viaduct governs as well the seismic structural responses, that provides an optimum protection against earthquakes.

It consisted of the following structural devices:

– Length wise unrestrained POT type bearings at piles P1 to P10 and P15 to P24. There were two on each pile in the transversal direction being the one free and the other guided in the longitudinal direction acting as seismic longitudinal isolators.
– Fixed POT type bearings at piles P11 to P14.
– Hydraulic Viscous Dampers (HVD'S) placed between the deck and the longitudinal fixed restraint at both abutments.

The four main functions of a Seismic Isolating System are ensured as follows:

– Vertical load transfer: By using POT bearings.
– Horizontal flexibility in the longitudinal direction: ensured by the sliding POT bearings and the longitudinal elastic deflection of the fixed P11 to P14 piers.
– Horizontal flexibility in the transversal direction: ensured by the elastic deflection of the piers.
– Re-centring force: by means of the longitudinal elastic deflection of the fixed P11 to P14 piers.
– Energy dissipation: by means of the hydraulic dampers (only active in the longitudinal direction).

The dampers placed at both abutments have also an important action during the serviceable state. They are activated by the braking and traction forces. In this case, the dampers act as traditional impact transfers without any remarkable movements. As a result, a sixth performance is obtained.

Stiffness against serviceability forces (longitudinal horizontal forces): ensured by the impact transferring function of the hydraulic dampers.

The Hydraulic Viscous Dampers are steel hydraulic cylinders with two internal chambers filled with silicon-oil type fluid. The fluid dissipates energy by means of the viscous friction and temperature increments when passing from a one chamber to the other.

The main performances achieved are:

Allowing for low speed movements with no considerable resistance, acting as traditional joints against thermal and rheologic imposed strains.

Transferring of longitudinal serviceable forces (braking and traction forces) with no considerable displacements (impact transferring function), acting as traditional fixed points against this type of actions.

Dissipating energy and limiting the transferred force to a desirable value when the earthquake takes place.

The first function allows for thermal and rheologic movements.

The second function prevents from short-term movements to happen under serviceable conditions.

4.2 *Description of the double steel-concrete composite-action strict box solution*

The basic ideas studied for a preliminary design phase presented by the authors at the 3rd International Conference on Steel-concrete Composite Bridges (2001) were developed to construction project level, which was finally built [4]. The French twin-plate-girder with a bottom steel truss bracing solution was modified according to the Spanish strict box with twin-plate-girder and double-steel-concrete composite-action technology. The basic ideas included in the paper at that Conference are summarized hereafter.

Nevertheless some previous aspects must be highlighted.

- The much lower weight, 5 t/m, of the launched steel-concrete composite cross-section compared with the weight of the concrete cross-section, 20 t/m, and its great flexibility at the launching phase, drastically prevented from problems such as adjustments to the route bends, technical and economic drawbacks arising from the use of two launching parks and, especially, risks of unadmissible stresses and deflections due to uncontrolled friction forces at the top of the piers. As a consequence, the launched solution, whose advantages and good adaptation to continuous steel decks are well known, turned out to be perfectly applicable to the steel-concrete composite alternative. In this case it was clearly better than an assembly by hoisting or erection with cranes.
- The steel-concrete composite solution had a slightly lower deck self-weight. It permitted to benefit from a significant reduction in the foundation costs, seismic forces dissipation needs (due to a reduction of the vibrating mass during the earthquake), piles lengths (which is always a critical point in the deadline of the project).

Conclusions obtained with the comparative study above mentioned between the conventional concrete and the steel-concrete composite solutions, which was developed at a high level of detail so as to the accuracy of the results required by the choice [5], lead the contractor to propose the composite solution to the GIF (now ADIF) Construction Department. Technical and constructive advantages, and less time required for execution were the main aspects considered in the election.

The project of the composite bridge was developed by IDEAM S.A., with several specialists in its team under the leadership of the authors of this paper.

Since there are a significant number of piers around 90 meters high, whose foundations and columns greatly condition the total cost and deadline of the structure, a preliminary study on launched steel-concrete composite solutions showed that an additional saving in both costs and deadlines could be obtained by using a series of standard spans of 63.50 meters, slightly larger than the 63 meters span sequence of the Orgon Viaduct, the greater span in the world up to now with this typology, located at the TGV Meditérranée Railway Line.

A launched steel-concrete composite solution of constant depth and 63.50 meters spans series, with a total length of 1208.90 meters, was finally designed (Figs. 17 and 18) and it is today under construction.

The deck cross-section (Figs. 19 and 20) was made up of two 3.85 meters deep twin-plate-girders plus a top slab with a maximum depth of 0.41 meters coincident with the cross section axis. The result is a steel-concrete composite cross-section of a constant maximum total depth of 4.26 meters and an equivalent effective depth of 4.20 meters (slenderness of about 1/15). The double steel-concrete composite action solution (Figs. 21 and 22) allows for a slight increment in the usual slenderness of the French twin-plate-girder solution (1/14).

The bottom of the section is closed using 0.14 meters deep precast slabs. These slabs work as collaborating formwork for the bottom concrete at the hogging areas near the piers. They are of a minimum thickness of 0.25 meters at 11.90 meters from the pier axis and of a maximum thickness of 0.50 meters at the pier axis. The bottom concrete represents the double steel-concrete composite action and allows for a maximum bottom plate thickness of 40 mm, much thinner than that of the French solution.

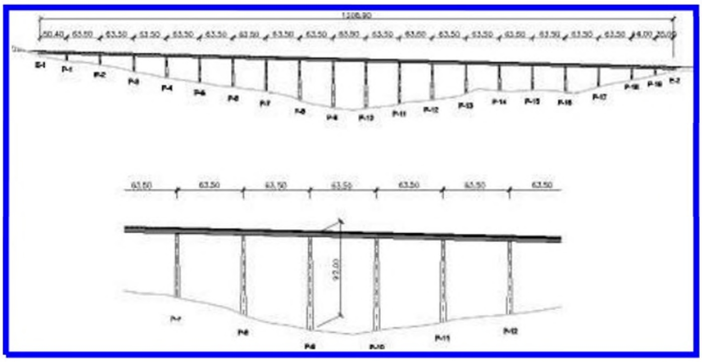

Figure 17. Bridge over Arroyo Las Piedras. From view of central zone.

Figure 18. View of the bridge over Arroyo Las Piedras.

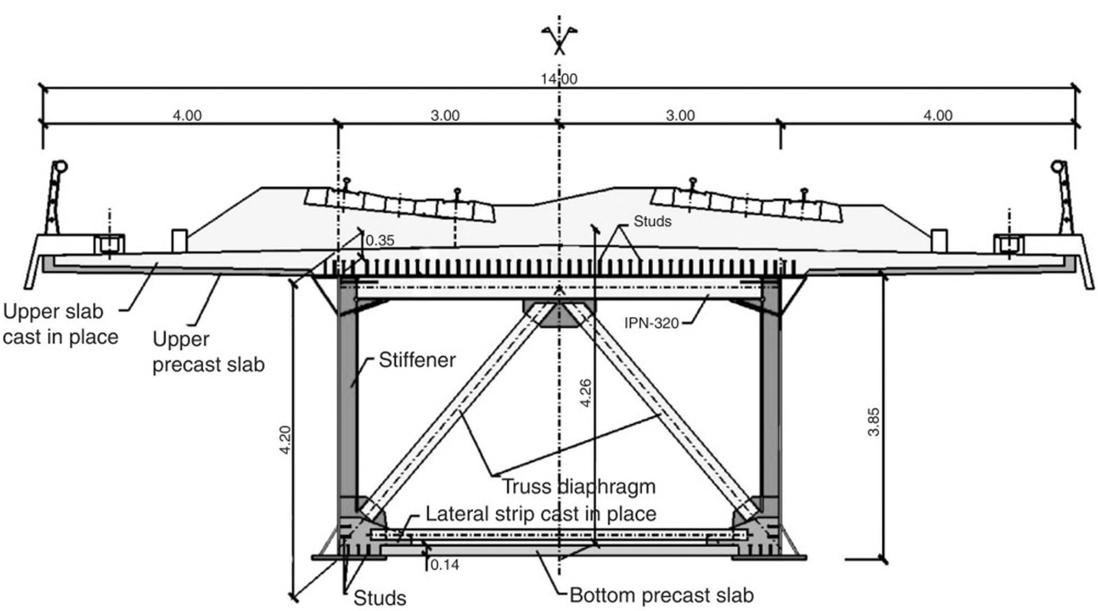

Figure 19. Mid-span cross-section. Steel-concrete composite solution.

Figure 20. Cross-section at mid-span areas.

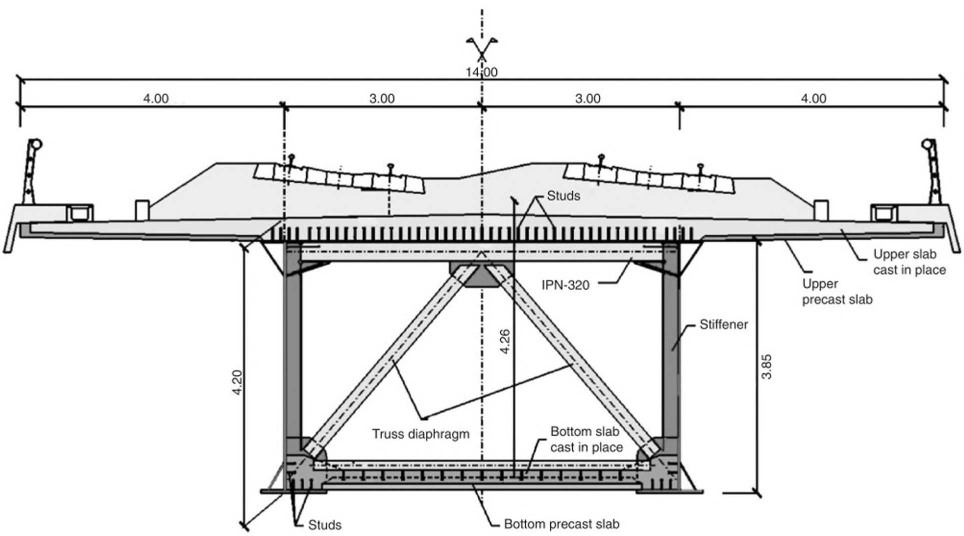

Figure 21. Hogging cross-section. Steel-concrete composite solution (63.50 meters span).

No bottom concrete is cast at mid-span (Fig. 19). The bottom precast concrete slabs close the torsion circuit for the strict box section. The need of closing the torsion circuit arises from limiting the warping strain of the section when there is just one train running over the track. This limitation prevents from leaving the mid-span cross-section area open as on road bridges, where it is really economical.

The 2.00 meters wide bottom precast slabs are not lengthwise connected among themselves. As a consequence, their collaborating effective width for sagging moments is quite low. Only a 1 meters wide strip at both sides of the slabs is needed to ensure the shear force transfer among the precast slabs and to control the crack width by using the required reinforcement. This cracking appears due to the restraint they exert on the steel flange tension under the action of the dead and live loads, which is also limited by the fatigue control at serviceable state of the tensioned steel plate.

Figure 22. Double composite action at hogging area.

Figure 23. Precast slabs.

The flanges of the lateral twin-plate-girders are not symmetrically designed in order to allow for the casting of the continuous concrete lateral strips. The triangular cells are positioned just at the ends of the section. The bottom precast slabs, which close the torsion circuit under sagging moments and act as collaborating formwork for hogging moments, give an external appearance of a continuous closed box. The bottom of the box combines two lateral steel strips with a central concrete slab.

The idea of allocating the cells outside the bottom plate arises from preventing water from accumulating at the external flange to web connection. It could be a point for the corrosion to start. Moreover, there is no possibility for the birds to refuge, which would be another way of corrosion.

The inexistence of steel bottom plates in the strict box section solution avoids using bottom stiffeners on the compressive plate which represent a great cost of steel and labour. The lateral plate-girders stiffening is reduced to the vertical webs consisting on stiffeners every 4 meters, easy to put in place in the factory. Strength requirements during launching are critical for the design of these stiffeners.

The top deck slab is made in two phases (Fig. 23). It is constituted by a lower precast slab with the same width than the deck platform and is divided in segments about 2 meters long in the longitudinal axis of the bridge. The total thickness of the slab (35 cm) is completed once the bridge has been launched, with concrete cast in place. Rectangular holes are modularlly distributed along precast slabs so as to allow the shear forces transmission required between studs and the slab, and reinforced bars are disposed to guarantee the appropriate connection between the precast units and these with cast in place concrete.

Precast slabs are laid over transverse steel profiles modularlly located along the bridge each 2 meters, that were made to collaborate with the slab spanning transversally between the webs under the dead and live loads. The rolled IPN 320 profiles were supported by primary vertical stiffeners every 4 meters and by secondary ones midway between the former. The top flange of the profiles includes studs welded to introduce composite action in transverse bending of the slab.

The transversal ribbed steel-concrete composite slab solution permitted to reduce the slab thickness to 0.35 meters regarding the variable thickness obtained by extrapolation of the concrete section, ranging from 0.33 meters between webs at mid-span to 0.50 meters at the top of the webs.

The vertical trusses are positioned every 8 meters approximately. They are made up of rolled profiles spanning from the centre of the transversal profile of the slab to the bottom corners of the section. The connection between those diagonals and the top profile is completed by welding a gusset plate. The welding can be done in the factory.

The steel used for the principal girders is a weather resistant steel (CORTEN type) with a yield strength of 355 N/mm^2. The thickness ranges from 25 to 40 mm both for the webs and the flange plates. The steel of the top slab transversal profile is painted having a yield strength of 355 N/mm^2. The steel of the transversal trusses is rolled and painted, having a yield strength of 275 N/mm^2. The concrete of the top slab is HP-35, that of the top slab bottom box is HP-50 and that of the bottom part of the double steel-concrete composite-action strict box is HP-40.

4.3 *The double steel-concrete composite-action strict box structural form in comparison with the standard twin-plate-girder solution*

As already described, the final solution constitutes a hybrid form of the French twin-plate-girder and the traditional in our country double steel-concrete composite-action strict box. The designed solution permits to benefit from the most important advantages of both systems as well as to eliminate the mainly economic disadvantages due to the current relative concrete/steel costs in Spain. This last point is more deeply described in the references [1], [3], [2] and [9].

Figures 19–22 describe strict box girder solution. Figures 24 and 25 illustrate a cross-section and a lower view of the typical French solution for the High Speed Lines.

The main differences from the designed solution are the following:

1. The plate web diaphragms of the French solution, of the same depth as the primary plate-girders placed every 6/8 meters and with holes to allow entry for inspection, are replaced with rolled profiles vertical lattices every 8/10 meters approximately. These elements guarantee enough stiffness against distortion of the section under eccentric loads and have a series of additional advantages:
   - Significant reduction of the total steel weight, around 15 kp/m^2.
   - Great simplification of the primary plate-girders on site assembly thanks to the described details, allowing for an easy adjustment of the transversal tolerances of the connections.
   - Reduction to a minimum of the transversal welds on site all along the web height, replaced with welded joints or High Resistance Bolts between gusset plates and rolled profiles.
   - Reduction of on site control requirements and slight improvement in the fatigue response of the detailing.
2. The web upper/lower stiffening in the French solution is substituted by some external triangle-shaped cells that link the upper/lower plates and the adjacent web areas.

   It is thus possible to:
   - Eliminate the visible horizontal surface of the external/lower semi-flange. This is the propitious place for the beginning of corrosion as it is to accumulating water and organic detritus, [3] and [2].

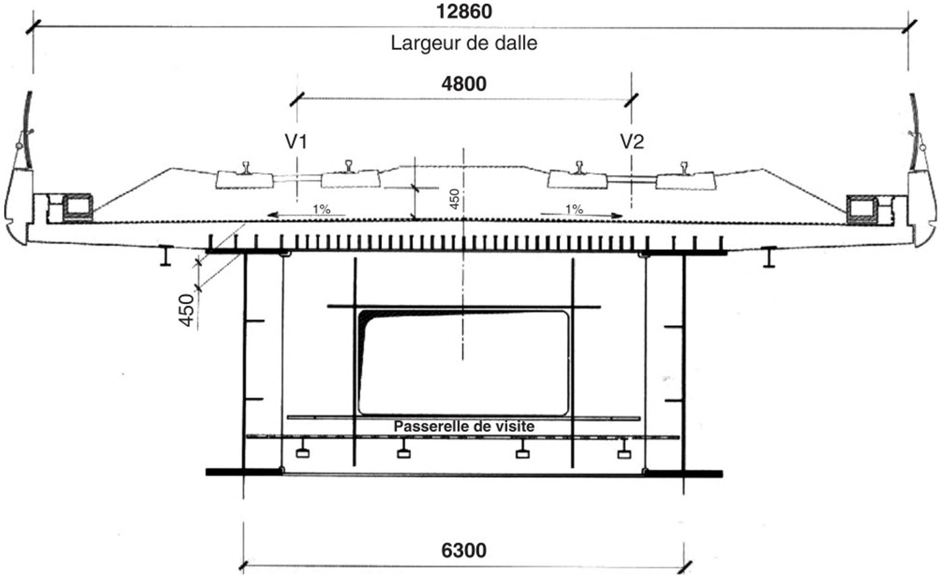

Figure 24. Classic French twin-plate-girder cross section for H.S.L.

Figure 25. Bottom view of the twin-plate-girder.

– Susceptible of keep the structural advantages of the lateral lower/upper cells typical of the Spanish double-steel-concrete composite-action box systems, that is to say the compressive web and flanges response against instability in assembly and serviceable states, [1], [3] and [2].
– Collaborate as tension/compression area with the adjacent flanges allowing for a reduction of their thickness to commercial values in our country.
– Reduce the local instability problems in the lower web part (patch loading) and the deck bottom flanges. This could be a dimensioning condition in certain areas due to local instability problems of the plates when passing over the pier supports during the launching phase.

The detail described is easy to make semi-automatically in factory, has good fatigue characteristics and helps to bring closer the main second moment of inertia axis to the web vertical plane. The possible alternative of a double triangle-shaped cell symmetric to the web vertical plane presents some additional resistant advantages. However, it was not chosen due to constructive optimisation reasons in the detailing of the site connections of the internal transversal bracings between primary twin-plate-girders.

3. The double steel-concrete composite-action strict box form turns to be more efficient than the use of bottom horizontal steel lattices to close the torsion circuit from the point of view of the torsion stiffness. The latter system is required in the French solution to limit the bending and torsion dynamic accelerations under the high-speed wagons running over just one track. The bottom slab thickness of the box in compression at the hogging area guarantees equivalent steel plate thickness from 37 to 73 mm. The 14 cm deep precast stabs present a highly controlled bending cracking at the sagging area. The reason for this is the tensile stress limitation at the bottom flange lower than 1100 kp/cm^2 under dead loads and 2950 kp/cm^2 under live loads, as well as the important continuous longitudinal reinforcement placed on site in the concrete strip adjacent to the steel webs. The absence of transverse connection between the 2 meters wide plates greatly prevents from the development of a tension collaborating effective width in the closure concrete and allows for low levels of cracking under longitudinal bending in those plates. The torsion stiffness study at the bottom flange of the strict box was done by means of a precise FE analysis. The results brought to an equivalent uncracked plate thickness of 16 mm reduced up to a minimum of 1.6 mm under the conservative hypothesis of total cracking, obtained by an approximate tie-and-strut analysis. In all the cases the global values of the torsion stiffness were important ranging from 80/250% at mid-span and 400% at the top of the piers, greater than those obtained with the horizontal steel lattices, which have an equivalent thickness around 0.8/2.0 mm. The double steel-concrete composite-action method enhances the stiffness difference at lateral span areas where torsion stresses are greater due to the double support of the deck onto the piles.

   The strict box solution has the following additional advantages:

   - Reduction of the total steel weight, around 15 kp/m^2.
   - Elimination of the delicate site welding operations between the bottom truss and the bottom flanges in the primary twin-girders.
   - Improvement in the structural fatigue response due to the elimination of such connections, as they require a careful design and execution to obtain the adequate fatigue strength.
   - Speeding up of the launching rates on site.
   - Elimination of the footbridge for inspection, necessary in the French solution. The closure concrete solution turns to be the better inspection pathway for erection and service phases of the structure.
   - Reduction of the double transversal wind front of the twin-plate-girder solution to only and wind front in the strict box solution. As a consequence, there is an important reduction of the transversal bending in piers and foundations due to their significant height.
   - Internal environmental protection of the section. Moreover, since the section is visitable there is an important reduction of the corrosion problems of the internal steel members.

4. The transversal IPN 320 profiles support the steel profiled sheeting formwork of the slab every 2 meters and are connected with studs once the slab concrete is hardened. The result is a steel-concrete composite frame acting as a rail support. A slight steel overweight of about 12 kp/m^2 permits an essential reduction in the concrete and steel ratios of the top slab and its weight. It is reduced from the 45 cm of the French solution to the 35 cm of the final one.
5. From a structural point of view, the main difference lays on the double steel-concrete composite-action of the strict box solution. As a result:

   - It drastically improves the inertia and dynamic stiffness conditions keeping a constant depth regarding the twin-plate-girder alternative, whose hogging stiffness is lower than its sagging stiffness. In our case the stiffness ratio is about:

     - (gross section at supports) $\risingdotseq$ 2.40 I+ (mid-span section) Due to cracking and tension stiffening, it falls to a maximum in the most stressed pier section of:
     - I (cracked section at supports) $\risingdotseq$ 1.30 I+ (mid-span section)

- It considerably reduces the positive bending and, therefore, the overall flexibility of the structure. With an approximate slenderness of 1/15, the static deflection limits established for the high-speed railway lines are widely fulfilled:
  - lateral span maximum deflection (UIC train) $\cong$ L/2500
  - central span maximum deflection (UIC train) $\cong$ L/2100
  - supports maximum rotation $\cong$
    - 3.200×10^{-6} radians (piles)
    - 2.000×10^{-6} radians (abutments)
- It drastically improves the torsion stiffness of the structure as shown in the dynamical analysis later on.
- It nearly eliminates the instability problems of plates in the ULS both at critical support sections and the mid-span sections. Class 1 or 2 sections are obtained for positive and negative bending. The ultimate curvature permits to reach ultimate strains of about 0.2% at the most compressive fibre and tensile maximum strains greater than 0.9%. An important reliable ductility for positive and negative bending is obtained in such way in the ULS. As a result, a ULS control of the type (Global Elastic Internal Loads/Elasto-Plastic Resistance of Sections for positive or negative bending) can be applied. There is even enough capacity for reaching situations of the structure close to the global plastic ULS by means of an adequate control of the elasto-plastic rotations and without any risk of brittle plates instability phenomena.

  This turns to be an irrefutable structural advantage of the strict box solution regarding the French twin-plate-girder alternative. The instability of the lower part of the web and bottom flange plates at the supports do not allow to take benefit from the structural advantages of the elasto-plastic transferring and dissipation phenomena of the steel-concrete composite structures in the ULS. It can even question the elasto-plastic response of the mid-span sections under sagging but for a detailed and precise justification, [12]. This response can be limited by the important restriction of the admissible elastic rotations of the support sections without risk of non-controlled instability.
- It all brings an important saving of the steel ratios and the stiffening of the primary twin-plate-girders. It can be evaluated as about 30 kg/m^2 for the bottom flanges at supports and about 10 kp/m^2 for web and flanges at mid-span. For this project, the maximum plate dimensions have been used:

  2 plates 800×40 mm at top flanges
  2 plates 1000×40 mm at bottom flanges

  regarding the 90 to 120 mm thick plates of the typical twin-plate-girder alternative. The latter solution forces the use of thermo-mechanical steel, whose supply is expensive in our country, delicate be to inspected in butt welded connections between thick main plates and difficult to be controlled by the site quality assurance.
- It also optimises the deck deflection response to imposed thermal and rheologic shortening/dilatation effects. Due to the typical viaduct length of the high-speed railway lines, considerable reductions in the joints movements of the railway expansion systems are obtained, which are weak points for the security, comfort and maintenance of the railway line, because:
  - the decks are not prestressed so that they do not suffer creep shortenings, which are the most important ones.
  - the concrete shrinkage is restrained and limited by the presence of structural steel.
  - the steel dilatation is restrained and limited by the bottom and top slabs, which have a greater thermal inertia and lower thermal expansion ratio.

  The average movements obtained would be:

  Creep: 0.0 mm/m
  Shrinkage: from 0.1 to 0.12 mm/m
  Thermal: from 0.4 to 0.5 mm/m

  Which gives place to a joint movement of, approximately, 60% of the traditional values for the prestressed concrete solutions.

Figure 26. Bridge during launching.

Figure 27. Bridge during launching.

– From a constructive point of view, the strict box solution has no important disadvantages regarding the French twin-plate-girder. The position of the steel section neutral axis is close to the box bottom slab, in the case there is such a slab. This makes that the bending alternation during launching produces low tension values in that slab without any cracking on it. As a result, launching can be executed on the whole steel section with the bottom slab completely cast, although another alternatives are possible for the choice of the constructor. The upper precast slabs are also launched with the structure except for the first zone with cantilever response during the launching. However, precast slabs are not connected to the steel beam, which is made after the launching with cast in place concrete.

  It is important to highlight that the launching section remains ready to visit and accessible during the whole launching process (Figs. 20 and 22), made simultaneously from both abutments.

– Reference [5] includes the results of the analysis of the fatigue and dynamic response under live loads. Improvements introduced by double composite action were clearly showed by the

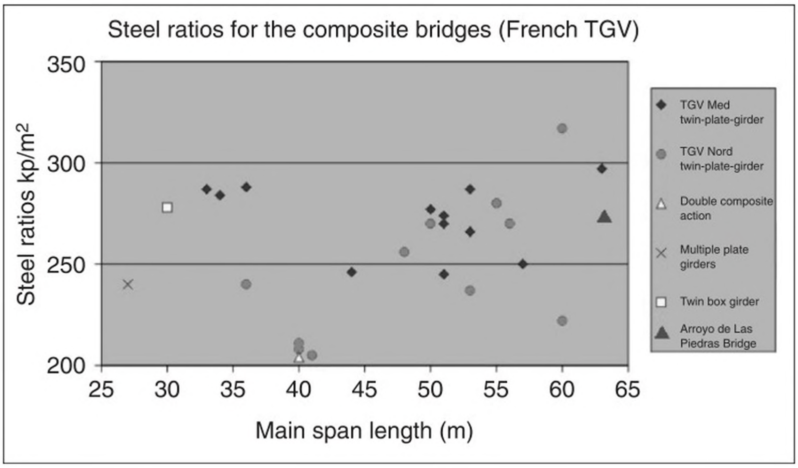

Figure 28. Steel ratios.

analysis in comparison with the twin plate guider typology, as a consequence of the higher torsion and bending inertia of the strict box.

Reference [6] includes some aspects related with structural response of the bridge in launching. Lateral strips cast in place together with botton cells and flanges constitute a major composite system to provide a safe launching process without web instability phenomena.

– Finally, Figure 28 illustrates some ratios obtained out on about 24 steel concrete composite decks of TGV Nord and Méditerranée. The solution was a continuous steel concrete composite twin-plate-girder. The steel ratios per m^2 of the platform range from 240 kg/m^2 and 290 kg/cm^2 for spans ranging between 30 and 60 meters at the main span and concrete slab thickness of 0.40 $meters/m^2$.

The long viaducts with 50 meters spans have constant steel ratios of about 270/280 kg/cm^2 reaching 295 kg/m^2 for the Orgon viaduct, with 63 meters spans. About 30 kg/m^2 less are obtained with strict box girder solution at Arroyo Las Piedras Bridge.

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CHAPTER 18

Engineering the bridge over the Hollandsch diep

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ABSTRACT: The structural design of the composite high speed railway bridge, with a 1190 m continuous multiple river span over the Hollandsch Diep, was the first of this type in The Netherlands and proved to be a real challenge. To fulfil the Design and Construct contract, severe demands related to aesthetics, tolerance requirements, ship collision forces and comfort criteria had to be satisfied in order to allow high speed trains to pass the bridge smoothly at 330 km/hour maximum speed. In the tender-phase various design solutions and erection methods were proposed in order to meet these criteria. In this presentation the alternatives that were evaluated in the design phase to obtain acceptable dynamic properties of the bridge structure will be shown and discussed, followed by final choices that have been made for structural design and erection.

1 INTRODUCTION

The bridge is part of the ±16 km long HSR-cluster from Rotterdam to Moerdijk which on his turn is part of the ±100 km long HSR-link that is being constructed between Amsterdam and the Belgian border. The Client is the Dutch Ministry of Transport, Public Works and Water management, represented by Rijkswaterstaat. The main contractor for the infrastructure is Bouwcombinatie HSL Drechtse Steden v.o.f., a joint venture of Ballast Nedam, Van Hattum en Blankevoort, Strukton, HBG, CFE, TBI, Vinci, and Van Oord ACZ.

The engineering of the bridge was done by IHD, a joint venture between IV Infra and Ingenieurs-bureau Rotterdam: hereby IHD was integrated in the design-organisation of the main contractor. The construction, transport and assembling of the steel superstructure of the bridge was done by the subcontractor, a joint venture of HSM, Hollandia and Mercon.

The main contractor has been responsible for the structural development of the largest high speed railway (HSR) bridge between Amsterdam and Paris. Since the bridge is considered to be a landmark for entering The Netherlands an explicit design was requested. Benthem & Crouwel was the winning architect of the design competition and showed a slender design with a multiple span composite girder having high V-shaped members at the supports and a reduced member height in the fields, see Figure 1 on next page.

The new piers had to be positioned in the shadow of the old 1868 multiple span railway bridge piers on the east side, the 105 m spans were synchronized. Apart from this, the new bridge is different in every aspect. The continuous composite structure proved advantageous in stiffness requirements, erection speed, maintenance and noise reduction.

The river part consists of 11 piers, about 105 meters apart, and 2 side spans of 70 meters, adding up to a total length of 1190 meters. In cross-section the structure consists of a 6 m wide an 4.5 m high open steel box girder, supporting a 14.2 m wide concrete deck slab with a maximum thickness

Figure 1. HSR bridge over the Hollandsch Diep.

of 0.5 m at the webs. In the support-region above the piers the height of the composite deck girder is increased to 10.5 m. The outer webs of the steel girder are 1 to 20 skew upwards, reducing the girder width by 1 meter at the lower bearing position. The bridge is supported by four rubber bearings at every pier and two bearings at the abutments. The bridge is longitudinally supported by its three middle piers and laterally at each pier by dowels made of welded heavy steel plates. To enable the large temperature deformations, sliding bearings at the outer piers and on the abutments support the bridge.

The piers consist of a cast in situ concrete shaft standing on prefabricated concrete casings. The casings are placed over large diameter steel piles and the rigid connection is made by in situ concrete. The approach bridges have a length of 400 m each. The approach bridges are constructed of concrete Π shaped members with a length of 32 meters each. The concrete members are designed in such a way that the side shape of the river span continues and slowly seems to descend into the ground.

The contract-based technical criteria related to comfort, stiffness and tolerances could hardly be met. This resulted in three different design approaches for the bridge during tender phase. The contractor had to optimize and integrate different views on construction methods and the structural designs for the bridge-deck and the piers. Therefore in final design phase a re-evaluation was done resulting in optimal choices out of the available solutions. The three different tender designs will be presented and the choices will be explained and discussed. The design of the bridge piers and approach bridges will be presented.

2 TENDER PHASE PRELIMINARY DESIGNS

2.1 *Tender 1*

The first design consisted of prefabricated 105 m sections, cantilevering 52.5 m symmetric from the piers. The 2400 ton composite parts are fabricated at the steel yard and transported to the building

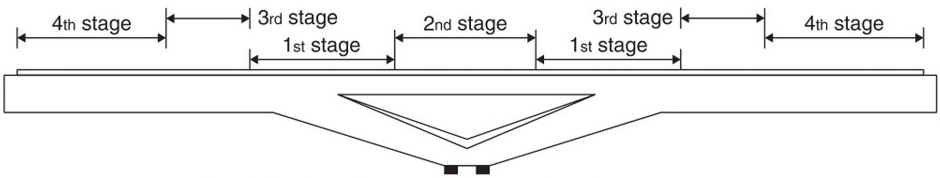

Figure 2. Casting order of the bridge deck with prestressing after the first three stages.

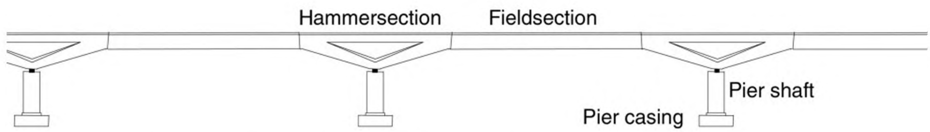

Figure 3. Bridge-sections are divided into hammer-sections and field-sections.

site on two barges. The height of the pier-section is 11.5 m which is too high to pass under the existing road bridge, the headroom being only 10.9 m. Lowering the bridge-section into the water while passing the bridge solved this problem. The section is installed on the pier and stability is achieved by temporary supporting frames and a rectangular 4-point bearing. This is a major structural change of the original design by Benthem & Crouwel, which had bearings, placed in one line.

The concrete deck was longitudinally pre-stressed over the pier-section, and transversely over the entire length, see Figure 2, to reduce the tensile stresses in the concrete and to allow full composite action for train loads. In order to direct the pre-stressing force in the concrete deck instead of the steel girder, the dimension of the steel upper flange and web had to be minimized. The concrete was cast in multiple stages, starting in the middle and going outward symmetrical in both ways from that position. After each stage pre-stressing force was applied with post-tensioning cables. In this way two goals were achieved: composite action in the negative bending moment region over the piers and a reduction in steel weight.

2.2 *Tender 2*

This design consisted of 105 m long composite and prefabricated parts, cantilevering 52.5 m symmetric from the piers, as did the first design. The concrete was cast in different stages, this time starting on the outside and working to the middle from both sides. The advantage of this method was that in this way there was no pre-stressing needed. The concrete was pre-stressed by the construction method itself. This method consisted of a progressive construction order in which each installed part was used to introduce a constant positive bending moment in the next part of the composite beam. The extra steel needed in the girder for this action was preferred over the use of post-tensioning cables.

2.3 *Tender 3*

The third design divided each span into two different sections, the 59 m long field-section en 46 m long symmetric cantilevering pier-section, also referred to as hammer-section, see Figure 3. The reasons for this partition were the transportation of the sections from the yard to the building site and the restricted accessibility of heavy lifting equipment at the site.

The hammer-sections are transported to the site without concrete deck. In order to pass under the neighboring road-bridge, the steel sections are fabricated and transported while lying on one side. In one action the 500-ton sections are rotated to the vertical position and placed on the piers with a floating crane. Temporary support frames are used to provide stability during construction.

The field-sections are prefabricated with concrete deck on the yard. On site, the 1200-ton field-sections are lifted from the barge with strand-lifts that are placed on the hammer-sections. After installing and welding the concrete deck of the hammer-section is cast. This reduces tensile stress in the concrete deck in the pier region. To reduce the amount of secondary steel the bridge structure was installed in three building phases. Between these phases the temporary support frames were dismantled and re-used in the next phase.

3 THE FINAL DESIGN

Though the third tender design proved to be the most efficient, dynamic behavior needed to be improved. During the final design the bridge superstructure has been optimized and details have been simplified. The bridge substructure has been re-evaluated and finalized.

3.1 *Comfort criteria*

The comfort criteria demands were difficult to meet since the combination of slender shape and severe equivalent maximum acceleration levels of 0.7 m/s^2 were nearly impossible to combine. Therefore the quest for activating additional structural stiffness dominated detailed design. The 105 m spans appeared to be unfavorable given the high speed of the train of 90 m/s, which means that the train passes a pier every 1.2 seconds, the characteristic frequency of the railway carriages being between 0.8 and 1.2 seconds. Together with the eigen-frequency of the structure in the corresponding range. These stroking frequencies result in an increased acceleration of the train. Improvement of the bridge characteristics in this aspect can be done by increasing the stiffness, by changing the mass of the composite girder, by changing the distance between the piers or by adding more stiffness to the supports. The distance between the piers was no real choice. Changing the mass of the bridge is difficult, since the dynamics showed better behavior with a lighter bridge deck. Apart from that, the unknown rail system had a great influence on the mass of the bridge and could not be influenced.

An option for adding stiffness to the structure was activating the stiffness of the piers. Pre-stressing the concrete deck on the hammer-section proved not economic due to the chosen construction method. In spite of the reduced stress in the concrete deck in the negative bending moment region, the concrete on the hammer-sections was cast only after the field-sections were installed, use of tension stiffening (according to Eurocode NVN-ENV 1994-2, annex L) did not result in additional stiffness. Because of the triangular shape of the hammer-section, the deck girder acted as tension member as well as a bending member, causing tensile forces in the concrete deck.

Incorporating the stiffness of the pier into the superstructure could influence support stiffness. The four-point bearing support at every pier added longitudinal stiffness to the bridge girder by introducing a semi-rigid connection to the pier. This improved the dynamic behavior considerably.

3.2 *Field section structural aspects*

The structural steel has been optimized. Therefore the use of buckling stiffeners is limited to 1 through-shaped stiffener at the 50 mm bottom plate and one at the 22 mm web plate, see Figure 4. The upper flange is 700 mm wide in order to have the same (500 mm) visual outside flange as the cross section over the piers. Every 5 m the cross section of the girder is braced with transverse T-shaped stiffeners 0.5 m high. The B55 concrete deck slab is poured upon formwork made from permanent hd-galvanized Comflor-70 steel sheet plates. The deck slab varies in thickness from 400 mm in the middle to 500 mm at the web-location and 200 mm at the outer edge.

3.3 *Hammer section structural aspects*

The hammer-sections are divided into 4 different structural parts: a deck girder, 2 Y-shaped connection pieces, 2 V-sections and the support section, see Figure 5. The open box composite deck

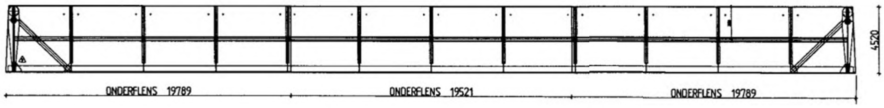

Figure 4. Side view of fieldsection.

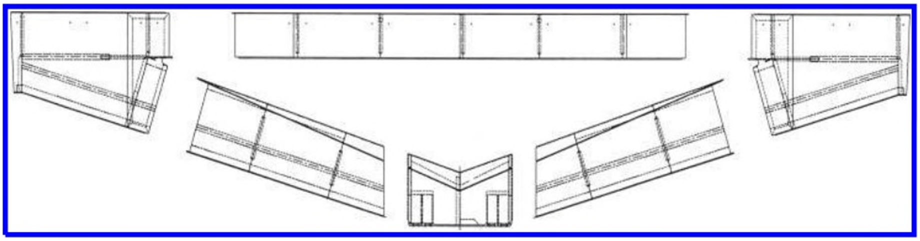

Figure 5. Side view of hammer-section, divided in production parts.

girder has a reduced height of 3 m for architectural reasons. Because of the triangular shape of the hammer-section the deck girder acts as a tension member as well as a bending member. Despite the construction method to reduce the tensile forces in the deck, composite action cannot be guaranteed in ultimate limit state. Therefore it's steel plate dimensions are larger than the field-section. The bottom flange thickness was increased to 60 mm because of large holes for the lights that are integrated in the bottom of the girder.

The Y-shaped connection pieces transfer the forces from both the field-section and the deck girder of the pier-section with the box girder of the V-section. The inner fold of the Y-section required special attention. Forces from the bottom flange are led into the web by a horizontal stiffener that was reduced from the middle to the webs. The complex junction between the 5 connecting plates was realised with a sliced piece of steel which is 5.5 m long and 800 mm wide.

The largest concentration of forces acts on the web plates. Extensive studies have been carried out to reduce the fatigue stresses in the inner fold of the web plate. A high quality steel was used, S460, with a thickness of 100 mm. After discussion with the architect the inner fold of the web plate was rounded in order to reduce the stresses. The connection between the 100 mm web plate and the 25 mm thick neighboring web plates was carried out with a 1:4 slope near the inner fold and 1:2 at a larger distance from the fold. As the outer surface of the web had to be smooth, the increased thickness on the inside of the web required all stiffeners to be adapted locally.

The steel V-box girders have high torsional stiffness to provide sufficient lateral support for the deck girder. The faceted upper flange of the box girder is made out of three steel plates. Every 5 m a brace frame was used to retain its form. The bottom flange is 50 mm thick, the webs 25 mm and the upper flange plates 30 mm.

The support region of the hammer-section is subject to large bending moments and compressive forces due to the triangular shape of the hammer-section. Three transverse plates are used to stiffen the 5 sided cross-section and one longitudinal plate is used to strengthen the upper flange. At the bottom flange, the bearings introduce a large concentration of forces into the support section. Limited space on the top of the piers required a fixed position of the bearings on the piers. Temperature differences of the 1190 m long bridge cause a longitudinal shift of 0.25 m over the bearings in both directions. In all possible positions the support forces have to be led into the steel. Heavy steel plates have been used to reduce the complexity of the detail resulting in a 150 mm thick bottom plate and three, max 80 mm thickness, vertical plates to transfer the forces to the webs. All welds are full pen.

3.4 *Support system*

The bridge bearing system changed as design proceeded. Starting with two vertical pot-bearings in line these were changed into rubber-steel plate bearings and finally changed to four bearings in

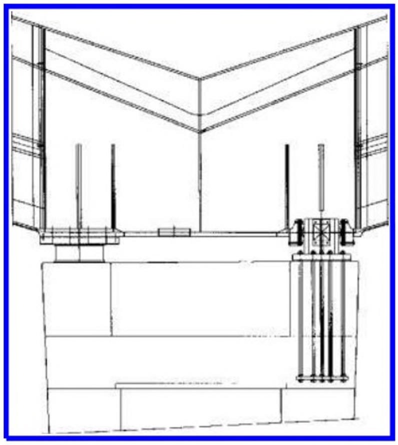

Figure 6. Position of vertical bearings and horizontal fixation on top of pier.

rectangular system. Latter was done to improve comfort-related stiffness of the superstructure since this position allowed moment transfer to the pier. By applying four bearings vertical flexibility had to be increased in order to keep positive reaction in all supports in case of a-symmetric mobile loading. Bearing dimensions are typical 1200 * 900 * 280 mm. Except for the 5 central piers all bearings have stainless steel-Teflon sliding plates.

After welding of the bridge-sections, the four permanent bearings on the piers are installed at their final level to obtain the requested equal load distribution.

All horizontal restraints are executed by means of a heavy steel dowel bolted to the top of each pier, see Figure 6. The dowel penetrates though a rectangular opening in the bottom plate of the support section. The opening allows for temperature. At the position of the bottom plate the horizontal supports are designed as flexible rubber steel plate elements in order to be able to cope with minor deformations. Due to reduced stiffness of the piers in longitudinal direction, a single pier was not able to take all longitudinal forces within deformation limits. Therefore the bridge superstructure is supported at the middle three sections in the direction of the span. To allow for temperature deformation both supports adjacent to the central pier have been provided with 10 mm free space.

In lateral direction the bridge superstructure is supported at all piers. This support transfers part of the ship collision impact force to other piers thanks to the high torsional rigidity of the bridge. In this way deformation during collision could be kept within limits of the contract requirements.

3.5 *Bridge substructure design*

The foundation of the piers consists of 5 hollow steel, open ended piles for the middle spans and 4 piles for the spans near the banks. With a length of 40 m and 3 m diameter these are driven into the bottom from above the water level deep into sand. This system proved to be effective over concrete piling as steel piles have large lateral stiffness and strength. Research was executed to improve axial stiffness by means of pressured grout injection at the pile bottom. However since this method was not applied before in similar situations and the operational risks were considered severe the method was skipped in favor of an increased depth of piles.

On top of the piles 25 * 10 m prefab concrete caisson were installed, see Figure 7. The rigid connection between the piles and caisson is made by means of casting under water concrete inner top of the pile and the caisson. The design of piles and bending connection were governed by the design criterion of a ship impact of 3000 ton acting 3 meters above waterline and allowing a maximum lateral displacement of 8 cm at the track level. After installation and sealing of the

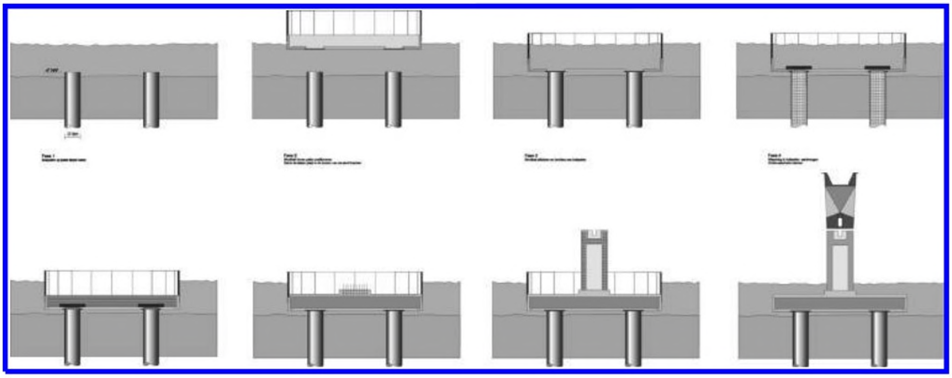

Figure 7. Building of the bridge piers in stages.

pile heads, the caisson was pumped dry. Since the top of the casing is 0.5 meter under water level temporary water retaining skirts were applied on top. Next the hollow casing is cast with concrete.

On top of the casing a hollow rectangular vertical shaft with varying cross section is cast in situ. The shafts have a varying height with a difference of 7 meters between the middle and the side piers. The shaft-heads include anchors for the bridge supports and therefore severe tolerances within mm range were applied for the shaft positioning. The mere 5 * 6 m top plane of the shaft included space for vertical supports at the edges, jacks for bridge lifting, horizontal supports and a central lower level inspection space accessible from the bridge.

4 BUILDING TOLERANCES

The rail system will be built by the infra-provider, a party that was unknown during the design of the bridge. The acceptable vertical tolerance of the deck was set at +/−15 mm, 1/7000 of the span. Considering the large spans, the continuous multiple span girder and the composite structure, these standards were hard to meet. In discussion with the HSL-ZHZ authorities this tolerance was set at +/−40 mm.

4.1 *Production*

The field-sections are built on the yard out of three 20 m long subsections. Though all spans are not exactly 105 m and the alignment didn't have a constant curve over the complete length of the main span, all field-sections were produced identically. The length of the middle subsection could be adapted in order to achieve the right length of the field-section. The subsections are set on hydraulic jacks in the right shape according to the alignment and camber. In this way the alignment curve was adaptable for every field-section. After welding and conserving the steel, the formwork for the concrete deck is installed and the concrete is cast. Because of the settlements of the soil the bearing positions are provided with an electronic measuring device. After casting the concrete the soil starts to settle. The deformation of the section is monitored continuously during the first days and after this period with larger intervals. If necessary the hydraulic jacks are used to correct the level. In this way the deformation is restrained to +/−5 mm. After a minimum of 14 days the sections are loaded onto a barge and shipped to the site.

The hammer-sections, all of them equal except for small details, are built on the yard out of 6 subsections. These parts are measured in the yard and the production tolerance has been set to +/−5 mm. At three positions, mirrors are on one side of the hammer-sections, two at the upper corners and one at the lower support region. The shape of the steel section, as measured in the

Figure 8. Erection of hammer-section.

yard, is related to these three mirrors which will be used to set the section on site. Since the site of the bridge is situated between two existing bridges the height of the transported parts is limited. Being 10.5 m high in upright position the hammer-sections have to be transported while laying on one side to reduce the height. This makes it impossible to apply the concrete deck in advance. Not earlier than on site the steel sections are erected.

4.2 *Erection*

In three building phases the hammer-sections are placed, four in the first two phases and three in the last. In order to be able to adjust the position of the section on the piers a temporary supporting frame is used. The hammer-section is set on hydraulic jacks, that is two on top of the pier and two on the supporting frame. During this operation, see Figure 8, the exact position of the hammer-sections is measured from the side of the existing railway bridge.

In between the hammer-sections, the field-sections are installed by strand lifts. For every hammer-section both adjacent field-sections are lifted and equilibrium is achieved. In this way the four bearings at each pier are equally loaded and divergence is minimized to $+/-10$ mm.

The first building phase takes place in the fall, the second in spring and the third in the summer. In this relative short period of time the total length of 1190 m is achieved. Between the phases, there will be a temperature difference. In order to minimise handling of the large and heavy field-sections, the position of the hammer-sections is corrected in advance.

After each building phase the formwork of the concrete deck is installed on the hammer-sections and the concrete deck is cast. In order to reduce the tensile stresses in the deck an initial gap of 2 m is left open between the prefab deck of the field-section and the in situ deck of the hammer-sections. Additional reinforcement is needed in the gap to account for the tensile stresses in the concrete of this intermediate part of the deck.

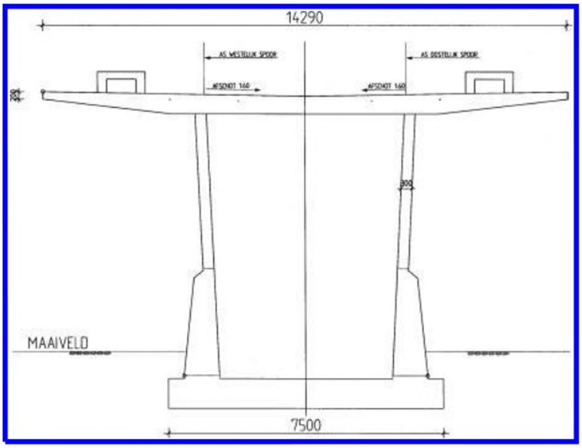

Figure 9. Π-shaped cross-section of approach bridge.

The settlement of the piers is monitored during building progression. Based upon these intermediate measurements the height of the hammer-section can be adjusted. In this way the strict allowance of the height of the deck can be met.

5 APPROACH BRIDGES

As the train may reach a speed of more than 300 km/h, the approaches must provide a smooth track to come from 20 m – NAP in the tunnel north from the bridge to 20 m + NAP on the summit of the bridge, in the middle of the river Hollandsch Diep. The alignment shows a maximum slope of 2.5% and is curved with a horizontal radius of 15 km. At the south end of the bridge the approach connects the bridge with the track that runs through the polder at a level of 3 m – NAP.

The height of the approaches varies continuously and all cross sections are different. The lower part of the Π-shaped cross section however is kept constant (Fig. 9) to obtain an easy to build foundation and ditto walls. Easy is a relative property, as the inclined faces (Fig. 10) of the lower wall required special adaptable formwork that was applied over the full length of the north and south approaches.

The part of the approach near the riverbank shows high walls. On land, south of the bank, the soil comes up to the level of the track. As the preference for the visible differences between parts inside and outside the Zwaluwse Dijk was communicated in a rather late stage, it was preferred to continue with one basic principle over the full length of the approaches on both sides of the river.

The main concern in the structural design of the approaches was the guarantee of a stable support for the track. For the river spans of the bridge the vertical stiffness was critical, however for the approaches the horizontal fixation appeared hard to obtain. The source of the problems is the settlement of the soil underneath and besides the structure and the deformation that follows from this. The vertical compressibility of the subsoil is high (up to over 1.5 m). At the east side of the approach there is the massive old dike of the existing railway and where vertical compression induces horizontal expansion of the ground, the old dike at the east side does not move, and all lateral displacements will go into westward direction. Several solutions and combination of them have been subject of study to reduce the horizontal displacement of the approaches.

Foundation of the structure on slurry walls didn't provide the acceptable results. Soil friction on both faces of these walls, in combination with the large height of the structure from lowest level (−20 m) to top level (+15 m) would cause too high horizontal displacements at track level due to the smallest settlements at the foot-level.

Figure 10. Side view of approach bridge, casting of concrete deck.

Braced piles appeared to have two disadvantages: the lateral movements would cause extreme loads on the piles and thus dramatically reducing the remaining bearing capacity for the structure. Moreover the vertical settlements would make the piles bend thus reducing the bearing capacity furthermore.

The possibility it was investigated if the structure could be corrected for horizontal movements by means of horizontal jacking. By making pile caps on vertical piles, a certain displacement cannot be avoided in that case, and by placing the approaches on Teflon sheet, it appeared to be feasible to move the last by means of hydraulic jacks in the opposite direction. The system would require reaction points to support the jacks and anchor points to correct and fix the position of the parts in the period between jacking. Since the Π-shaped structure is hollow, there is room to do these corrective works without influencing the trains on top of the deck. Anyhow it is a complicated solution. It proved to be effective that most of the movements took place before the structure was built Advanced and time consuming calculations for the prediction of the behavior of the soil made clear that almost all horizontal movements of the ground and the structure would take place before installation of the track if:

- a temporary over height and overweight was applied, and
- vertical drainage would allow the groundwater to migrate rapidly.

This solution was followed during the further detailing and construction of the approaches. Decrease of bearing capacity due to negative skin friction and bending of the piles were taken into account.

6 CONCLUSIONS AND DISCUSSION

Within a relative short period of time the final design has been developed. This was possible due to input of the main contractor in the evaluation of available solutions and the fact that the design team consisted of members of all three tender designs. The main challenges of the design were the severe comfort demands for a passing train in combination with the slender architectural design, the rigidity of the piers under the high collision forces together with the use of a low number of large steel piles and the limited time for the horizontal deformations of the approach bridges.

During final design the construction in the shop together with the erection activities on site have been optimized, leading to an overall high level of prefabrication and standardization and to reduction of actions on site. The steel girder has been optimized for the combination of a reduced amount of stiffeners and reduced thickness of steel plates. Use has been made of heavy steel plates with higher steel quality in order to avoid complex and labor consuming details. The thickness of the deck has been optimized and the use of lost formwork has appeared to be favorable. All hammer-section are identical and so were all field-sections. The last have been produced in a production line. The prefabrication of the concrete deck of the field-sections on the yard has reduced the work on site considerably.

Due to the construction method with the division of the erection in three phases, storage time was reduced for the steel and composite girders. The re-use of temporary supporting frames resulted in a limited amount of erection equipment.

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Sub contractor steel: Staal Trio Moerdijk
Photographs: IJ.J. Broekhoven

REFERENCES

[1] Design of the bridge over the "Hollandsch Diep" has been in accordance with the Dutch standards and supplementary specific HSR guidelines. Reference will be made only to these guidelines:
[2] Guideline HSL 500, rev. F. Technical design codes HSL-South.
[3] Guideline HSL 600, rev. E. Loads and deformations of civil structures.
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