13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2205

SEISMIC PROTECTION OF TUY MEDIO RAILWAY VIADUCTS: DESIGN AND SHAKING TABLE TESTS OF THE SEISMIC DEVICES

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ABSTRACT

This paper presents the seismic design of the Caracas-Tuy Medio Railway bridges in Venezuela, which is based on the use of isolators equipped with steel hysteretic dampers. All testing activities validating design concepts thereof are also presented.

The considered isolation system comprises two main elements: sliding bearings and steel dissipating components of the tapered-pin type. This system is characterized by complete independence of the vertical load carrying element from the steel dampers that control horizontal actions and dissipate energy.

In order to validate the design assumptions, a series of shaking table tests were conducted to verify the performance of the steel dampers used. The seismic input consisted of two sets of acceleration time-histories, artificially generated from two different design spectra, with peak ground accelerations of 0.41g and 0.37g respectively. Each set comprised four acceleration time-histories and these were consecutively applied to the same pin damper. The system used for the tests comprised four free sliding bearings, one full-scale pin damper and a rigid mass representing the deck. Furthermore, it also included a sacrificial restraint, generally used to impede any damper displacement under service loads. This is the first instance in the world where a sacrificial restraint/dampers system undergoes full-scale shaking table tests. It should be noted that the mock-up represents a portion of a typical bridge deck span. In effect, the steel damper used for the tests was a full-scale pin element identical to those installed. The modularity of the steel hysteretic dampers permits conducting full-scale shaking table tests without having to recur to scaled down magnitudes. In fact, the model mass corresponds to the portion of the structural mass isolated by a single component and there is no need to do any time scaling.

INTRODUCTION

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Between 1980 and 1990, the government of Venezuela, through its Autonomous State Railways Institute (IAFE), decided to begin construction of a railway system for the central region to eventually link the capital, Caracas, with the port of Puerto Cabello on the Caribbean. The initiative issued from the need to link the main cities of the central and central-occidental regions through a people-freight transport medium other than public highways that can prove safe and efficient. In fact, present usage of the existing highway network far exceeds it capacity to support such volume of traffic. Inevitably, traffic congestion leads to an obviously progressing increase in travel time as well as costs. As a reflection of this, the project aims to greater development of the areas create touched by the line and decentralize urbanization demands to decongest in part the greater Caracas urban area. The construction of the new railway system, where the Caracas - Tuy Medio tract is only the first phase, represents the grandest railway development project in the South American Continent.

TECHNICAL CHARACTERISTICS OF THE RAILWAY LINE

The line linking the capital to the Valle del Tuy along 40 km crosses a prevalently mountainous area, covered by tropical vegetation comprising the farthest extension of the Andes chain, characterized by deep ravines through which run water torrents whose flow is extremely variable (quebradas). The railway line is thus a continuing alternation of tunnels and viaducts. These structures necessitate the observance of the project parameters basic to the project conception so as to guarantee both speedy connections as well as an acceptable level of passenger comfort.

The project consists of the construction of 14.9 km of tunnels and 7.8 km of viaducts respectively, throughout the 40 km extension of railway line. All the 26 multi-span viaducts are provided with seismic isolators and railways joints manufactured by FIP Industriale (in total 217 spans; the longest span has a length of 95 m).

As it regards project design loads and level of passenger comfort, it calls for applicable UIC international norms [1].

AREA SEISMIC CHARACTERISTICS

The Caracas – Tuy Medio Railway, is located in Venezuela´s north central region, whose seismicity is influenced by the interaction of the Caribbean and South American tectonic Plates.

The Seismic Hazard Study (Grases, [2]), conducted for this project, established a “Design Earthquake” with 7.5% probability of being exceeded during the expected service life (70 years), which is equivalent to an average return period of 900 years (annual probability of exceeding = 0.11%). For damage control, the study selected events (“Verification Earthquakes”) associated with an average period return of 50 years (annual probability of exceeding = 2%), which are probably to occur more than once during the service life of the structure.

Ground motions are defined by the characteristic parameters and spectral shapes indicated in the Seismic Hazard Study from which the elastic response spectra are built for each component (horizontal and vertical) in the area in question and corresponding subsoil profile. The mountain viaducts of this railway are founded on grounds essentially comprising metamorphic rocks, mainly graphite schist and phyllite. Essentially two types of acceleration spectra, corresponding to two different soil types, were used for the design, characterized by a PGA of 0.41g and 0.37g respectively (Figure 1).

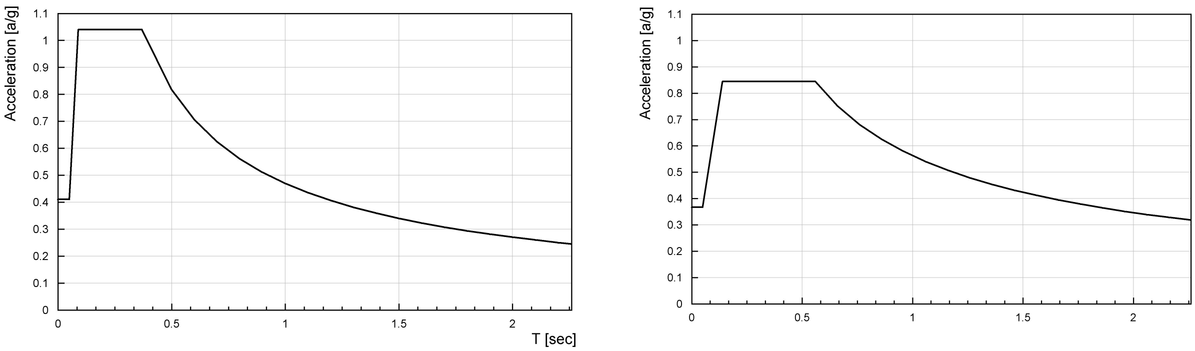

Figure 1. Elastic acceleration spectra type AST (left) and type IMP (right).

DESIGN CRITERIA

The selection of the bearing devices and the type of passive energy dissipation elements for the seismic isolation of the decks was based upon the functions that they should comply with to guarantee the proper operation of the trains under normal and emergency conditions, as well as under exceptional conditions which might be produced by a severe earthquake. To accomplish this, the following basic principles were established:

- To provide an isostatic deck-substructure configuration due to the presence of geological faults across the viaducts' course.
- To avoid uncontrolled deck movements during normal and emergency train operations, basically for reasons directly related to the support of the rails.
- To avoid the occurrence of structural damage under the conditions imposed by the "Verification Earthquake".
- To protect the structural elements (in particular the piers) from failure under the conditions imposed by the "Design Earthquake".
- To protect the decks from losing support when seismic displacements associated with the "Design Earthquake" take place.

The study of the seismic isolation device system was conducted by the FIP Industriale Technical Department, in cooperation with the design engineers. The study of the restraint system for both service and seismic conditions led to the choices described in the following.

Normal Service Conditions

As said above, under service conditions, the viaducts are composed by isostatically restrained decks. The isostatic configuration provides for bearings characterized by very high lateral stiffness to all the required operational loads, thus impeding major deck movements that might damage the rails and thus the line itself.

The proposed restraint system provides for the adoption of bearings with different characteristics.

For the steel bridge decks, said restraint system called for the following types of bearings (Fig. 2):
- a fixed device + a transverse guided one on the fixed end;
- a longitudinal guided device + a free one on the mobile end.

Seismic Conditions
The bridge deck bearing arrangement provides for all bearings to become free to move in all horizontal directions thus ensuring a high level of seismic energy dissipation (Figure 2). Control of maximum displacements and forces transmitted to the substructure is achieved through the use of special dissipating pin elements that will be described in the following. The failure during earthquake of sacrificial restraints, also described below, permits the change from the stiffly restrained configuration required for the service conditions to that for seismic conditions.

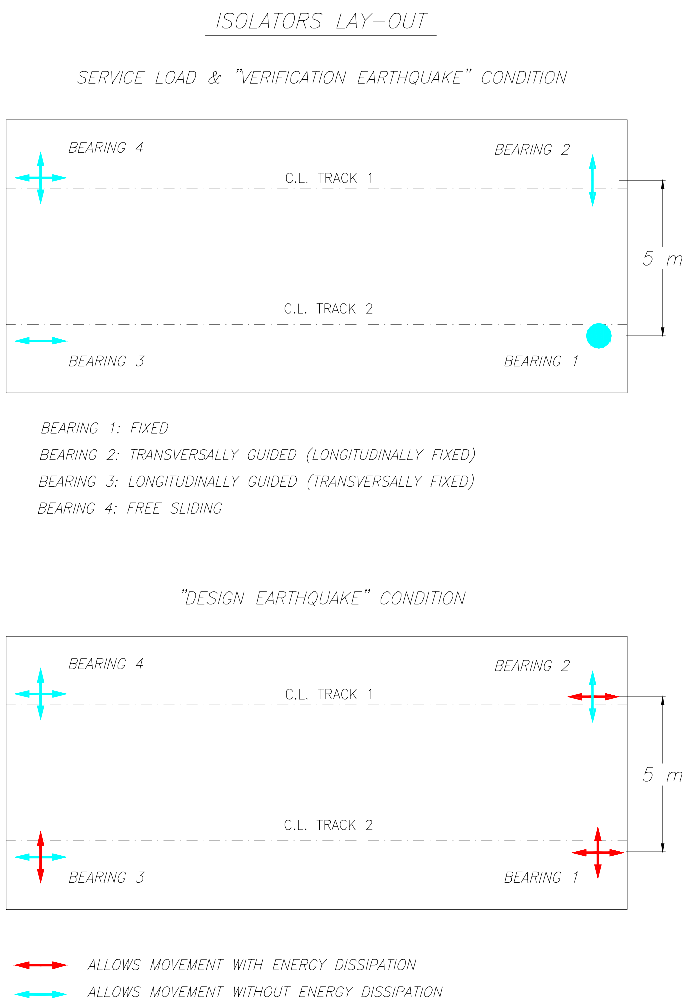

Figure 2. Typical devices lay-out in service (above) and seismic (below) conditions.

DESIGN OF THE ISOLATORS

The seismic isolation system selected for this project comprises three main components, the bearing, the steel hysteretic dissipating elements, and the sacrificial elements (fuses), in this case combined within a single isolator. The bearing serves to transmit vertical loads and allow horizontal displacements. The steel hysteretic dissipating elements dissipate energy and control the horizontal actions generated by the earthquake. The sacrificial elements are designed to resist service loads and to fail during the design earthquake. Dissipating elements can have different shapes, i.e. crescent moon, tapered pin, double tapered pin, etc., and different yielding mechanism of the steel elements (bending, torsion, etc.). Once the type of element and its correct size are selected according to design displacement, the steel dissipating elements are arranged to achieve the required function (uni-directional or multi-directional). The required yield and maximum force are obtained by setting the proper number of dissipating elements working in parallel. Thus, the isolator is characterized by the modularity of the dissipating components, which makes it easily adaptable to different structural requirements. This modularity also gives the advantage of redundancy: i.e., any defect in one or more elements does not put a seismic device out of service. Said system, also known as the Italian technology of seismic isolation, has seen many successful applications in viaducts, bridges, and other structures all over the world since the eighties. Some of the outstanding applications are the retrofit of the Marquam bridge, Oregon, USA [3], the Bangabandhu bridge in Bangladesh [4], the retrofit of the Granville bridge, Vancouver, Canada and the retrofit of the Chirag I offshore platform in Azerbaijan [5].

The steel hysteretic dissipating elements selected for this project are the tapered-pins, i.e. cantilever-type elements. The adoption of these axially symmetric elements guarantees characteristics of multidirectionality and isotropic behaviour in the horizontal plane.

The isolators (Figures 3 and 4) comprise the following main components:
- Vasoflon pot bearing (Maximum vertical loads from 3000 to 26000 kN)
- Sacrificial elements (fuses)
- Dissipating hysteretic steel elements (tapered pins)

The latter special components are described in the following.

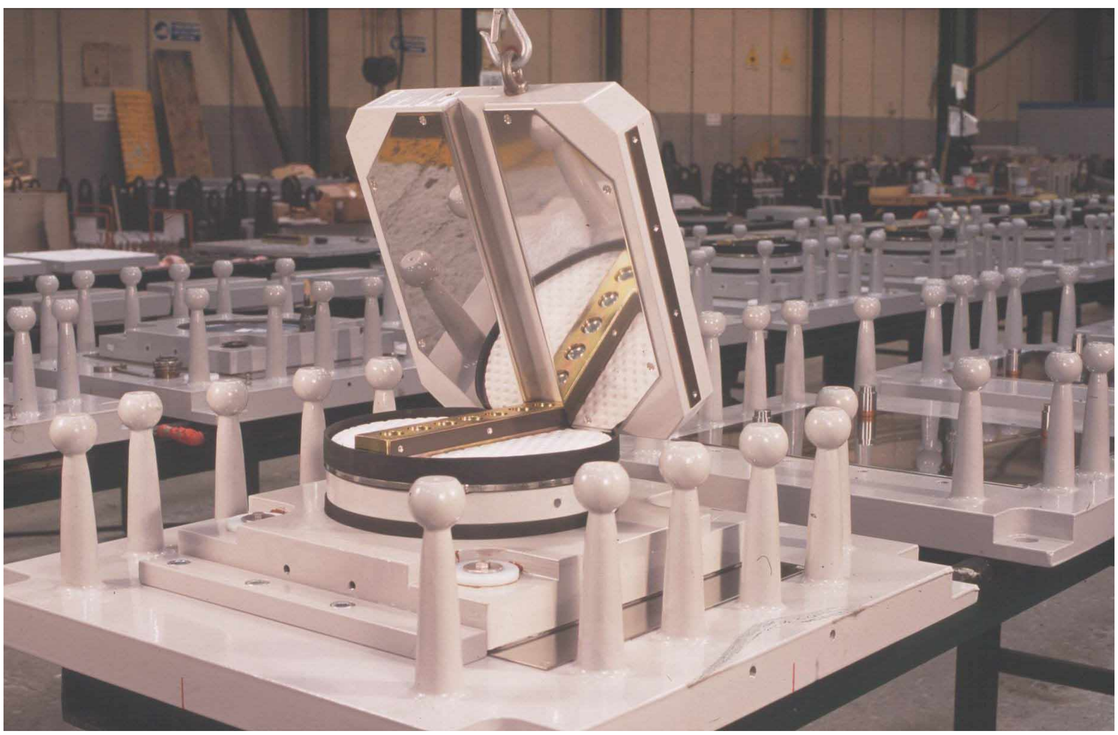

Figure 3. Isolator during assembling phase in workshop (type 2 and 3 in Figure 2).

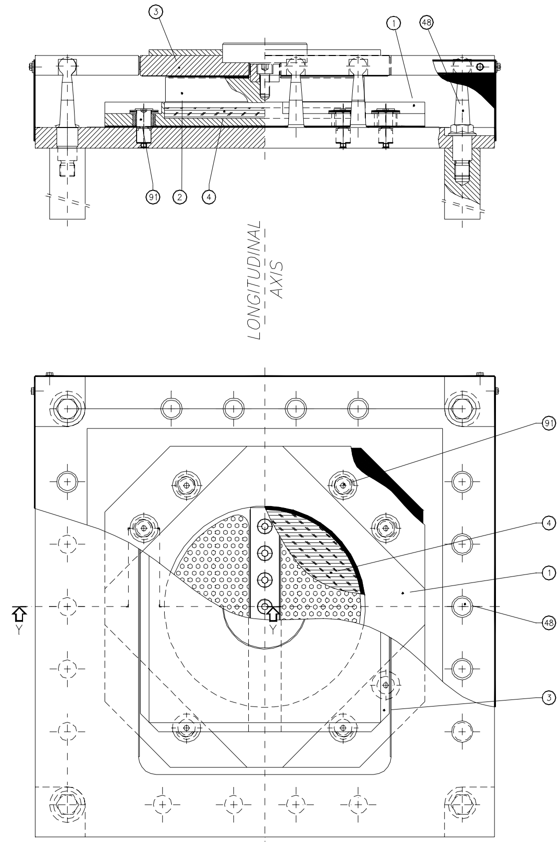

Figure 4. Guided seismic isolation device arrangement
1) Pot element; 2) Intermediate element; 3) Sliding plate;
4) Elastomeric disc; 5) Steel dissipating pin; 6) Sacrificial element (fuse)

Sacrificial elements

Sacrificial elements are used in combination with steel hysteretic dampers. They are designed to transmit horizontal service loads (i.e. wind, braking actions, etc.) to the substructure without significant displacement (that is particularly important with railway viaducts) and to fail during a design earthquake, thus allowing the steel hysteretic dampers to work (figure 5). Said elements are shear fuses that work elastically until the failure threshold is reached. When the fuses shear, the fixed and guided bearings become free, and keep the bridge deck restrained through the dissipating elements incorporated in the bearings themselves. The isolators positions no. 1, 2, and 3 in figure 2 are equipped with four or eight sacrificial elements.

Figure 5. Installation of a sacrificial element in an isolator.

Tapered Pin Dissipating Elements
The use of dissipating pins together with the special restraint system offer a practically uniform seismic protection to the structure independently from seismic direction.

The tapered pins are characterized by high dissipating efficiency, that is nearly 60%, which leads to an equivalent viscous damping equal to 40%. To achieve such performance, the dissipating devices are manufactured using special austenitic steels with high physical-chemical characteristics, inalterable in time and capable of resisting a great number of cycles at maximum amplitude, much higher than the number of cycles asked for in the project design.

The device hysteretic behavior is represented by a bilinear curve comprising an elastic as well as a plastic branch (for this part the increment in load is decisively reduced).

The constitutive behavior of the dissipation element can be modeled as a bilinear function. Figure 6 shows the idealized force-displacement hysteresis loop associated to the maximum design displacement, represented by M (d_{max}, F_{max}).
The intersection of line r_1 with the Force axis corresponds to the so called characteristic strength F_d.
The yield force is denoted as F_y, Elastic stiffness K_o and Post-elastic stiffness K_d.

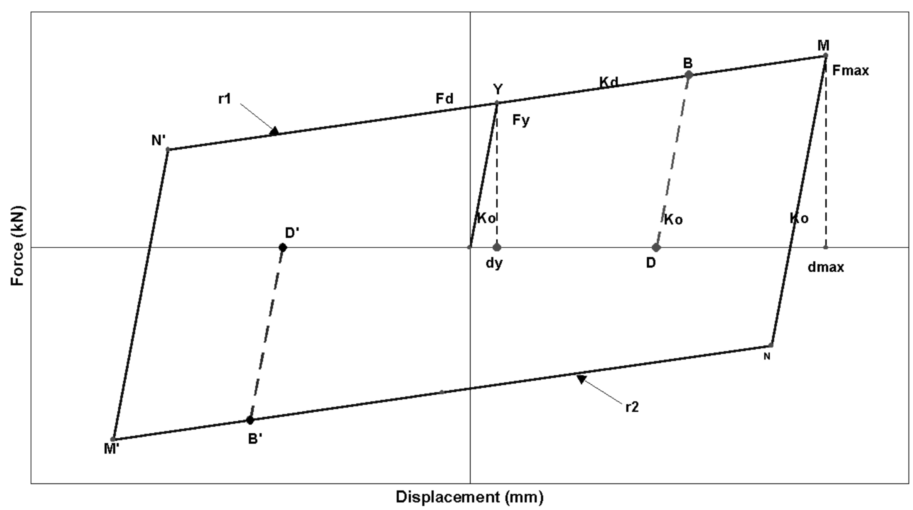

Figure 6. Force vs. Displacement model curve.

EXPERIMENTAL INVESTIGATION AT FIP LABORATORY

The dimensions of the dissipating elements depend on the horizontal force and on the maximum displacement, so different types of tapered pins were used in this project.
The properties of each dissipating element, as well as those of sacrificial restraints, were verified by individual tests. In particular the tapered pins were subjected to low-cycle fatigue tests at constant displacement (equal to the design displacement) at FIP Industriale Laboratory (figures 7 and 8).

The number of cycles that produces failure, when the specimen is subjected to maximum displacement, indicates that its service life can be over 20 cycles. Due to the fact that an intensive earthquake corresponds to only 3 ÷ 4 cycles at the design displacement in terms of energy dissipation, it will not be necessary to substitute them because their life should be longer than the life of the viaducts they have to protect.

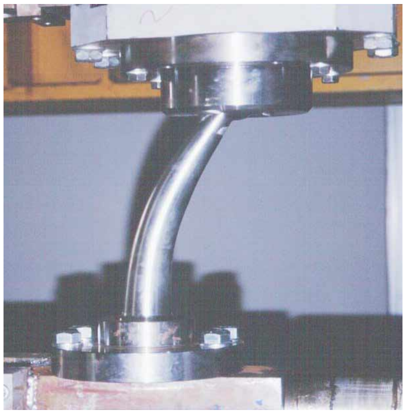

Figure 7. A single tapered pin dissipating element during tests at FIP Industriale laboratory

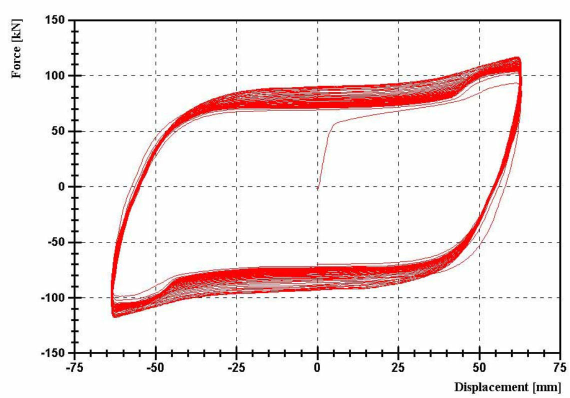

Figure 8. Force vs.-Displacement experimental curve of the smaller dissipating element

SHAKING TABLE TESTS

Test facilities and experimental model

The experimental program was carried out on the shaking table at the ENEL.Hydro (formerly ISMES) laboratories in Bergamo, Italy. The table measures 4 m $\times$ 4 m in plan. It can be controlled in 6 degrees of freedom, but the tests here described were carried out in only one horizontal direction, so as to simplify the evaluation of results.

The test structure was meant to be a model of the deck of two typical viaducts of the Tuy Medio railway line (viaducts No. V5-4, Group A and No. VI19, Group B). A typical span of said viaducts was considered as reference structure for the tests.

The scope of the tests was to study the dynamic behavior of the steel hysteretic dampers and not that of the deck itself. Thus, the model structure is extremely stiff and comprises a series of steel and reinforced concrete masses supported by a steel frame (Figure 9). It is considerable that the model isolation system consisted of a full scale steel hysteretic dissipating element, identical to those used in the isolators installed in the viaducts. It is the modularity of the isolation system described above that allowed not having to scale down the damper, as it is usually necessary with shaking table tests. Using full scale elements permitted to avoid scaling the time and all other quantities. Consequently, the tests results are more reliable and representative of the actual situation, when compared to tests performed on scaled-down models. The model mass thus represents the portion of the deck mass that is seismically protected by a single dissipating element in the isolated viaduct, i.e. the mass value given by the ratio of the total deck mass to the number of dissipating elements.

Two different mass values were used for the model, i.e. 38765 kg and 27760 kg, for models representing viaducts No. VI19 and No. V5-4 respectively. For example, the dampers installed in a typical span of viaduct No. VI19, having a length of 40 m and a mass of about 1240 t, comprise 32 tapered pin elements for each direction (i.e. longitudinal and transversal). Similarly, the reference span of viaduct No. V5-4, with a length of 30 m and a mass of about 1200 t, is isolated through 44 tapered pin elements. Thus, in this second case, the mass corresponding to one dissipating element is lower.

Figure 9. The test model.

The dissipating element is installed at the center of one side of the steel frame. It is characterized by a nominal yielding force of 43 kN, yielding displacement of 5 mm, a maximum design displacement equal to 65 mm, a maximum force equal to 57 kN and post-elastic stiffness 0.027 times the elastic stiffness. The model rests on 4 free sliding bearings. There are also 2 guiding devices, aimed at avoiding displacements orthogonal to the direction of the test.

The model isolation system, as well as the actual one, also comprises a sacrificial restraint designed to fail at 127 kN. This value was obtained dividing by 32 (i.e. the total number of tapered pins) the total design failure force (4050 kN) of the sacrificial restraints installed in the reference span of viaduct No. VI19.

It should be noted that it is unusual for the total failure force of the sacrificial restraints to be higher than the total maximum force given by the dampers. In effect, the service loads in a railway viaduct are much higher than those in an highway viaduct (where they are usually about 1/3 of the seismic load). Furthermore, the inertial forces induced by an earthquake on the piers are quite high in these viaducts. Thus, the total bending moment at the base of the piers due to service (quasi-static) loads is approximately the same as that dynamically induced by an earthquake, given by both the inertial forces on the deck controlled by the dampers, and the inertial forces on the piers themselves.

Nine channels were recorded during each test to measure the response of the combined shaking table-structure system. Two displacement transducers, installed at the southwest and southeast corners measured the relative displacement between structure and table. Another displacement transducer directly measured the top-to-bottom damper displacement (thus reducing the effect of gaps in the values measured). Two tri-axial accelerometers were mounted at the center on the table and on top of the structure respectively.

Experimental program

Two main series of tests were carried out. The first series was carried out on the model with a mass equal to 38765 kg. The model was subjected to a set of 4 different artificial earthquake inputs generated to match the AST type acceleration spectrum defined above, according to AASHTO specifications [6]. Some input was repeated more than once, so that the first series comprised 8 subsequent earthquake inputs (numbered from 1 to 8).

The second series of tests was carried out on a model with a mass equal to 27760 kg. The model was subjected to a set of 4 different artificial earthquake inputs generated to match the IMP type acceleration spectrum. Even in this case, some input was repeated more than once, so that the second series comprised 12 subsequent earthquake inputs (numbered from 9 to 20).

All the tests in each series were conducted on the same damper. The damper was substituted after the completion of the first series (test No. 8), before the start of the second series (test No. 9).

Before said tests, a further series of tests was conducted at lower intensity to calibrate the table control and account for the high non linearity of the isolation system.

Experimental results

The following discussion of experimental results mainly focuses on tests No. 1-8, i.e. those at higher intensity. Notwithstanding this, the results of tests No. 9-20 were similar.

Test No. 1 was carried out on the model comprising the sacrificial restraint. The time history plot of the deck accelerations measured during Test 1 (Figure 10) clearly shows the difference between the two phases of the test, before and after the failure of the sacrificial restraint that occurs at circa 3.5 seconds.

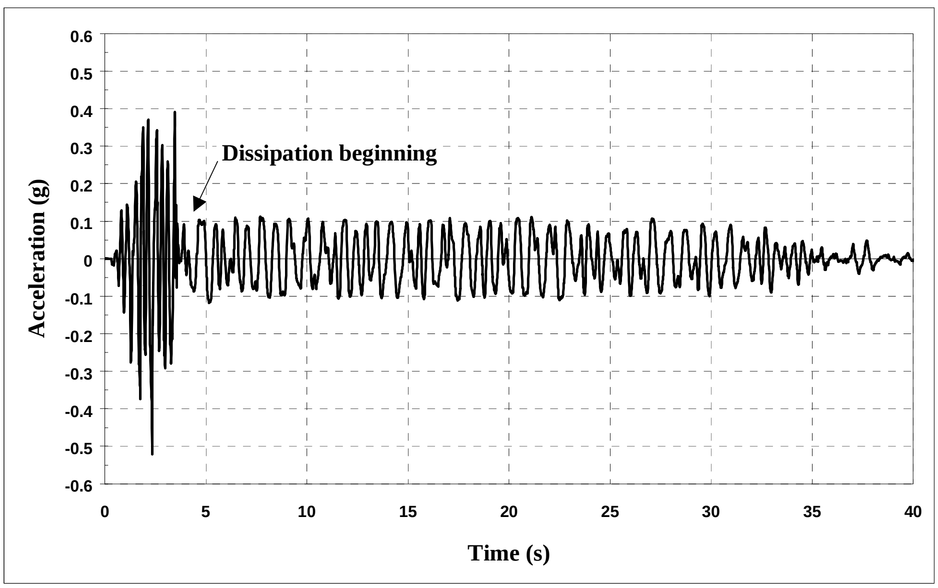

Figure 10. Time history of deck acceleration. Test No. 1, with sacrificial restraint.

The first phase is characterized by much higher accelerations (up to 0.51 g) and frequencies than the second phase when the damper is activated. It is also evident how the acceleration peaks remain almost constant (about 0.12 g) during the second phase, owing to the post-elastic behavior of the damper.

The hysteresis loops reported in figure 11 also show the different behavior during the first phase, with a very stiff response and higher forces, and the second phase, with the typical bi-linear behavior of steel hysteretic dampers.

Figure 12 shows a comparison between the first 6 seconds of the deck acceleration time histories measured during tests No. 1 and No. 2. The latter was carried out with the same input (AST4) as test No.1, but without the sacrificial restraint. After the failure of the sacrificial restraint, the acceleration response was identical for the two tests. This shows how the damper works properly immediately after the failure of the sacrificial restraint.

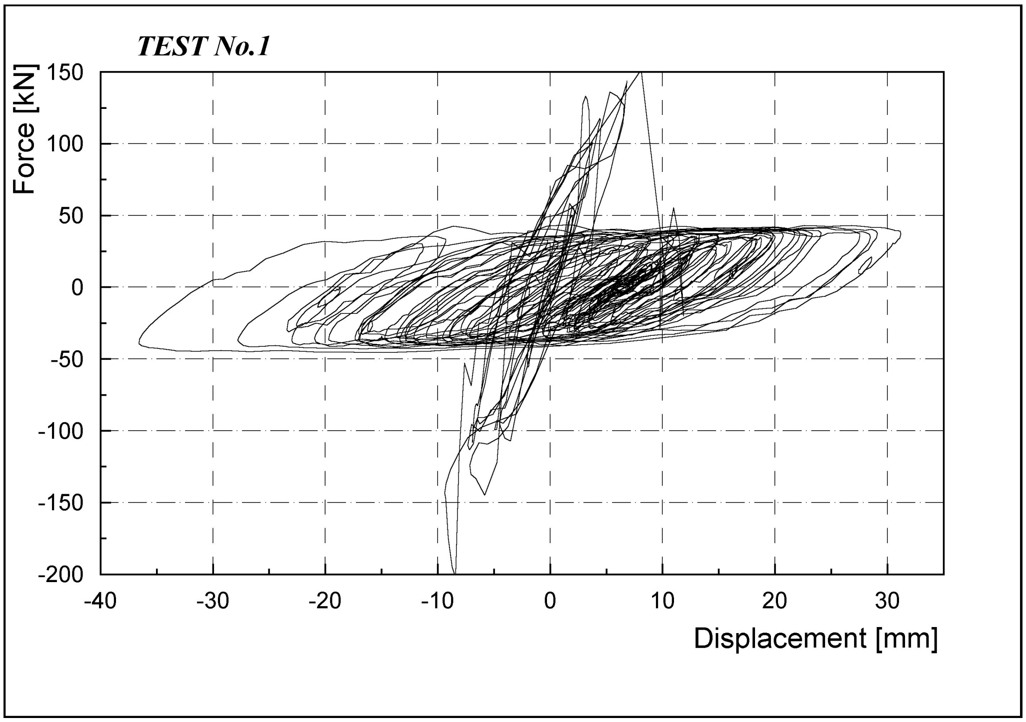

Figure 11. Hysteresis loops for the steel hysteretic damper. Test No. 1, with sacrificial restraint.

An impressive reduction of acceleration above the isolation system was observed during each test. The acceleration reduction due to the isolation system falls within a range of 51÷72 %. It can also be observed comparing the time history plots of table and deck accelerations (Figure 13).

Test results show that a steel hysteretic damper can withstand at least 8 maximum expected earthquakes. In effect, the first damper was subjected to 8 consecutive tests with inputs type AST without any damage.

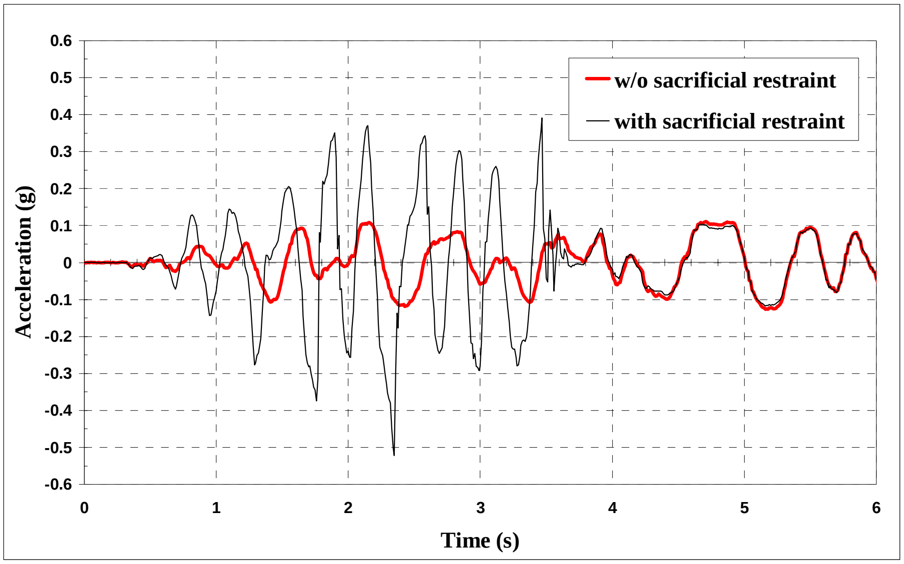

Figure 12. Time history of deck acceleration. Comparison between tests No.1 and No.2, with and without sacrificial restraint respectively, for the first 6 seconds.

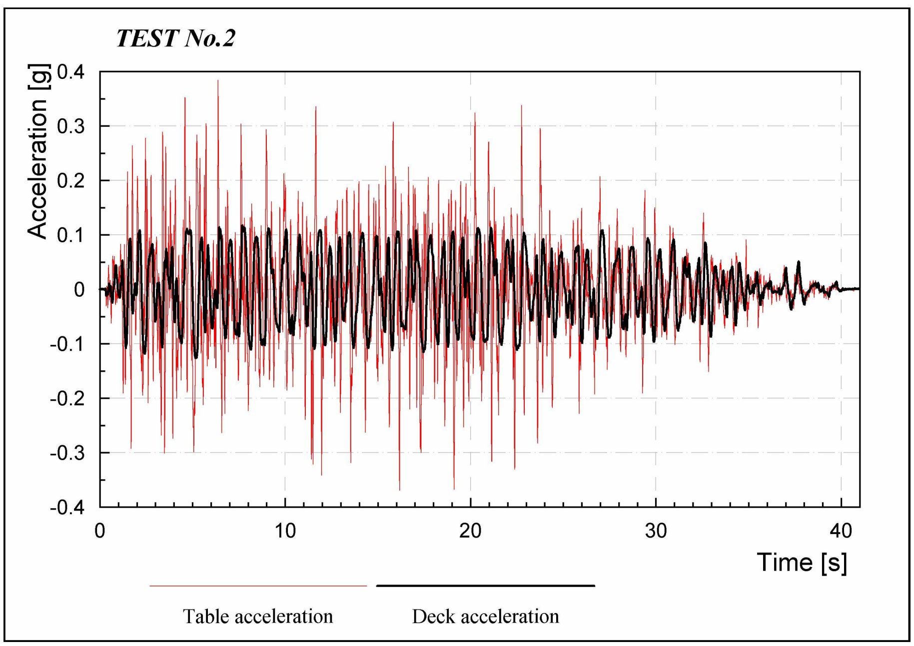

Figure 13. Time histories of table and deck accelerations. Test No. 2

The second damper was subjected to 12 tests with inputs type IMP, and the damage started only during the 12th test. This, together with the results of countless low cycle fatigue tests, confirms the fact that there is usually no need to substitute steel hysteretic dampers after the occurrence of an earthquake with the design intensity (in this case associated to a 900-year return period) and energy content.

A typical time history plot of the damper displacement is shown in figure 16, relating to test No. 7. It can be observed that the residual displacement is about 10 mm only.

The recorded responses during the test sequence performed on the same dissipating element indicated stable hysteretical behavior. The energy dissipated by the tapered pin during these tests was 42.5 kJ and 43.2 kJ respectively, despite the different input.

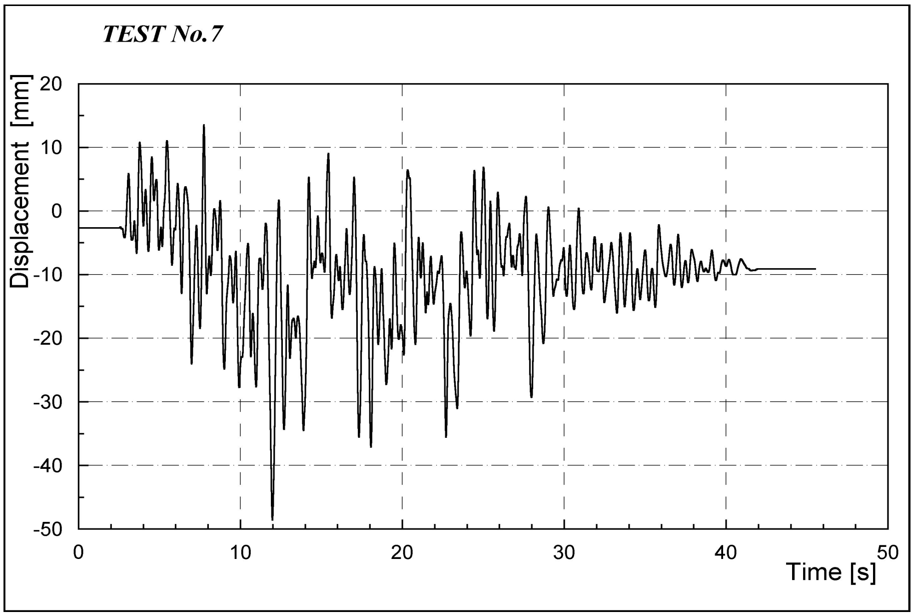

Figure 14. Time history of displacement of the tapered pin. Test No. 7.

CONCLUSIONS

By means of incorporating seismic isolators with energy dissipation devices between the deck and the substructure, it is possible to control the maximum inertial force transmitted from the deck to the piers as well as to limit horizontal displacements and associated costs. Passive energy dissipation is an effective way to protect structural elements against the damaging effects of an earthquake and it appears to be the technical and economical solution that satisfies the requirement for the adequate seismic performance of bridges (and other structures) in high risk seismic zones. The cost associated with the substitution of sacrificial elements after a design earthquake is much lower than that originated by damage and/or collapse of a non-isolated structure.

A full scale portion of an actual bridge deck was tested on a shaking table, owing to the modularity of the steel hysteretic dampers selected that comprise a number of dissipating elements working in parallel. The experimental model reproduces the portion of the deck mass corresponding to one dissipating element and is seismically isolated through a single full scale tapered pin acting as dissipating element, and sliding bearings. As far as these authors know, it is the first time that the behavior of a seismic isolation system with a sacrificial restraint has been dynamically tested on a shaking table. Test results validate the use of sacrificial restraints in combination with steel hysteretic dampers. In effect, after failure of the sacrificial restraint, the damper behaves exactly as it would have without a sacrificial restraint. Furthermore, test results show that the steel hysteretic damper tested can withstand at least 8 maximum expected earthquakes without failure. Results also show a significant reduction in superstructure accelerations due to the isolation system. During the different tests, the peak acceleration on the deck ranged from one third to one half of that on the shaking table.

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