RION-ANTIRION BRIDGE:
FULL-SCALE TESTING OF SEISMIC DEVICES

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Keywords: Viscous Dampers, Fuse Restraints, Seismic Protection System, Energy Dissipation

1 ABSTRACT

The Rion-Antirion Bridge, which crosses the homonymous strait in Greece, is located in a region prone to high intensity earthquakes and large tectonic movements generated by local active seismic faults. The deck of the multi-span cable stay bridge is continuous and fully suspended (for its total length of 2,252 meters) from four pylons (with spans 286m + 560m + 560m + 560m + 286m). The bridge is designed to withstand seismic events with peak ground acceleration of 0.48g (2000 years return period) and tectonic movements between two piers of 2 meters in any direction. This was possible through the use of an innovative energy dissipation system that connects the deck to the top of each pier and limits the movement of the deck during the specified earthquake, while dissipating the seismic energy.

The seismic protection system comprises fuse restraints and viscous dampers acting in parallel, connecting the deck to the piers. The restrainers are designed as a rigid link intended to withstand the high wind loads up to a pre-determined force corresponding to a value slightly higher than the wind load ultimate limit state. Under the specified earthquake, the fuse restraints are designed to fail, leaving the viscous dampers free to dissipate the energy induced by the earthquake into the structure.

The behavior and the requirements of the dissipation system were optimized with the help of a non-linear time history numerical analysis on a 3D model. It resulted on seismic devices (viscous dampers and fuse links) characterized by extreme design parameters in terms of force, velocity and stroke, with dimensions and weight never attempted before. The design assumptions and the actual behavior of the seismic protection system had to be confirmed with extensive testing of full-scale prototypes.

The aim of the present paper is to present the testing program and the results of the full-scale prototype testing. The Viscous Damper Prototype tests were performed at the laboratory of the University of San Diego California (USA), while the Fuse Restraints were tested at the FIP Industriale Testing Laboratory in Italy.

2 INTRODUCTION

The development of civil structures like bridges and buildings characterized by ever increased dimensions has led to a confrontation of very complex problems linked to the imposition of dynamic actions by natural events. Windstorms and earthquakes can in fact compromise the function as well as the safety of such structures.

In many cases, induced structural oscillations can be efficiently damped through the introduction of devices endowed with a constitutive force-velocity law (viscous dampers) that can furnish another capability to dissipate the energy associated with the aforesaid natural events.

In order to obtain the most efficient performance these devices shall be installed where the maximum relative movement between two adjacent structural elements is allowed.

The Rion-Antirion Bridge, located in an area prone to strong seismic events and windstorms, shall be equipped with a damping system that for its characteristics and for the strict design requirements entitles to put this structure in the next generation of seismic protected structures.

The seismic protection system comprises fuse restraints and viscous dampers acting in parallel, connecting the deck to the piers. The restrainers are designed to work as a rigid link in order to withstand the high wind loads. Under the specified earthquake, the fuse restrainers are designed to
fail, leaving the viscous dampers free to dissipate the energy induced by the earthquake into the structure.

Four viscous dampers (Fmax 3500 kN, Stroke ±1750 mm) and one fuse restrainer (Fmax 10500 kN) shall be installed at each pylons, while at the transition piers two viscous dampers and (Fmax 3500 kN, Stroke ±2600 mm) and one Fuse Restrainer (Fmax 3400 kN) shall be considered.

These viscous dampers are of ever built dimensions and design capacity and shall be installed at the pier to deck interfaces in order to reduce the transverse swing of the deck during a dynamic event.

They are designed to dissipate the energy introduced into the structure by the earthquake into heat.

Requirements for a proper and safe seismic behavior are sometimes not in agreement with the everyday service life of the structure. Thus, large structural displacements induced by moderate earthquakes or windstorms are avoided by an additional restraint system, that at the occurrence of a major design event fails allowing the structure free to oscillate with its damping system.

This system is composed by Fuse Restraints installed in parallel to the dampers, so that the deck, when subjected to lateral loads not exceeding their design capacity, is linked rigidly to the substructure, while after failure it leaves the deck free to swing coupled to the dampers.

Figure 1 gives the general arrangement of the four viscous dampers and the restraint device at one pylon. The dissipation system connects in the transverse direction the fully suspended deck to the pylon base.

![Figure 1 Rion Antirion Bridge: Arrangement of Viscous Dampers](image)

3 NON-LINEAR VISCOUS DAMPERS

FIP fluid viscous dampers are piston/cylinder devices that utilize fluid flow through orifices to absorb energy. Orifices are situated in the piston head, which allow the fluid to move back and forth between the two chambers. The force generated by these devices is the result of a pressure differential across the piston head. These devices are equipped with two spherical hinges at both ends that keep the transmitted load aligned along their main axis. This detail is of major importance to provide a reliable performance: it prevents the piston rod from bending and thus the sealing system from failing. High strength steel components have been used for the vessel and the plated piston rod so as to withstand the actions imposed by a dynamic load. The anchoring details depend only on the structure to which they are anchored: for example, the tang plate/clevis system illustrated in Figure 2.

In the last few years, FIP dampers have undergone major testing programs at FIP laboratory as well as at independent facilities [3] [11] (Earthquake Engineering Research Center, Berkeley CA – USA, Boeing Testing Facility, Canoga Park CA – USA and Caltrans SRMD Testing Laboratory at the
University of California – San Diego - USA), which has entitled the company to be a pre-qualified damper manufacturer for the Golden Gate Bridge Retrofit project as well as to enter the CALTRANS (California Department of Transportation) list of pre-qualified damper manufacturers. It should be noted that remarkable reaction stability has been achieved by dynamically cycling within a very wide temperature range (-40°C ÷ +50°C) that guarantees proper behavior under any type of environmental conditions.

A very important issue related to the utilization of the technology entails the correct numerical modeling of the devices as integrated into the structural model and the performance verification of the finished product. The latter topic will be amply discussed as follows.

The behavior and the requirements of the dissipation system were optimized with the help of a non-linear time history numerical analysis on a 3D model of the entire bridge. A parametric study on the effect of the characteristics of the dissipation system demonstrated that the structural integrity does not depend on the exact value of the non-linear exponent (\(\alpha\)). See Table 1 for the stroke and the force on the dampers for different cases of non-linear exponent. Despite this, strict specifications have been imposed with regards to the behaviour of the dampers in order to ensure a stable performance of all devices.

<table>
<thead>
<tr>
<th>Non-linear exponent ‘(\alpha)’</th>
<th>Maximum Dynamic Stroke (m)</th>
<th>Force (MN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>1.24</td>
<td>3.20</td>
</tr>
<tr>
<td>0.2</td>
<td>1.18</td>
<td>3.25</td>
</tr>
<tr>
<td>0.3</td>
<td>1.11</td>
<td>3.30</td>
</tr>
<tr>
<td>0.4</td>
<td>1.06</td>
<td>3.40</td>
</tr>
</tbody>
</table>

**Figure 2** Damper Main Components

### 3.1 Modelling

The most appropriate mathematical model to represent the behavior of an FIP viscous damper is to use a Maxwell constitutive law characterized by a linear spring in series to a non-linear dash-pot element (see Figure 3). The first element represents the elasticity of the system that is mainly due to the compressibility of the silicon fluid, while the second depicts its damping properties. The exponent that characterizes the non-linearity of the response as a function of the velocity, is particularly
important. Positive effects are undoubtedly related to low exponent dampers: e.g. in bridge applications, reduced pier deformation, lower expansion joint costs and cost effective bearings or in buildings, maximum reduction of relative displacement at the isolation interface or in the braces.

FIP viscous dampers are designed to guarantee a 0.15 exponent so that, within a wide velocity range, reaction is expected to be almost constant. This performance characteristic permits the devices to initiate their damping reaction at low velocities, providing the maximum reduction of the superstructure displacement.

Total system elasticity, mathematically represented by the stiffness $K$, depends on the compressibility of the fluid and it is particularly evident at the point of motion reversal, or at any rate, at the point of transitory phases: commonly, it is a secondary effect and often it is not even computed.

![Figure 3 Damper constitutive law equation and hysteresis loop](image)

To evaluate the response of such a system, the differential non-linear equation representing its constitutive law needs to be integrated.

Looking at the plot in Figure 3, it can be noted that this typical hysteresis loop looks almost rectangular even if the displacement time–history is sinusoidal (thus velocity dependent). This is due to the damper characteristic exponent (0.15). In fact, the behavior is almost independent from velocity. Thus total damper behavior can be similar to that of a spring in series to a “perfectly plastic” element characterized by a constant threshold force.

### 3.2 Viscous Dampers Full-Scale Testing

The following paragraphs present the main result of the testing activities performed at both FIP Industriale Testing Laboratory and at the Seismic Response Modification Device (SRMD) Testing Laboratory of Caltrans at the University of California at San Diego [11] on a full-scale prototype of the viscous dampers to be utilized on the above mentioned bridge.

The prototype, characterized by a 3500 kN reaction at the maximum design velocity of 1.6 m/s and by a ±900mm stroke, was submitted by FIP Industriale on February 2002 for evaluation.

This viscous damper is deemed to be the largest ever-built device of its type (6.4 m pin to pin) and was tested up to its maximum design conditions. The prototype is equal in every detail to the dampers designed for final installation with the exception of the stroke, which is shorter so as to fit into the existing test rig.

#### 3.2.1 Full-scale testing at FIP Industriale Laboratory – Italy

The above mentioned prototype underwent to two series of tests. The first one was performed at FIP Industriale Testing Laboratory, where the unit was tested for final tuning of its damping characteristics before shipment to the UCSD laboratory for official testing in presence of the client (Kinopraxia Gefyra - Greece) and of the bridge design checker (Buckland & Taylor - Canada).
FIP Industriale Testing Laboratory is equipped with a power system providing for 630 kW at 1200l/min thus resulting to be the most powerful in Europe for full-scale dynamic testing of seismic devices.

Being the above unit of exceptional characteristics, the prototype was tested up to the maximum velocity provided by the available system which resulted to be 0.2 m/s. In the following figure, the test results are reassumed and it is evident how the damper behavior follows the theoretical constitutive law with a minimal deviation.

![Prototype during testing at FIP Laboratory](image)

Figure 4 Prototype during testing at FIP Laboratory

![Damper Constitutive Law](image)

Figure 5 Damper Constitutive Law

In Figure 4, the prototype is shown during dynamic testing at FIP testing Laboratory. For testing purposes, the unit was pinned at one end to the test rig (yellow colored) while its piston rod was connected directly to the actuator through a 3000kN capacity load cell. The damper reaction was measured through the mentioned load cell as well as indirectly from the pressure detected on the actuator through a pressure gauge. The displacement was measured through a magneto-strictive displacement transducer (see Figure 5 for testing results).
3.2.2 Full-scale testing at SRMD Testing facility – California, USA

The damper OTP 350/1800 was tested at the Seismic Response Modification Device (SRMD) testing facility of the University of California San Diego. The test matrix is reported in Table 2.

The specimen was horizontally installed in the testing rig, connected at one end to a reaction wall and at the other end to the movable platen. The six degrees of freedom table is capable of unique level of displacement, forces and velocities. For this specific application the platen was moved in the longitudinal direction, with a displacement control loop able to maintain the components of motion in the other directions at negligible level. Figure 6 shows the damper installed on the testing frame.
A summary of results is presented in Table 3 for Thermal, Stroke Verification and Velocity Variation tests. Peak forces at different velocity levels are reported in Figure 7.

### Table 2: Test Protocol Prototype FIP OTP 350/1800

<table>
<thead>
<tr>
<th>Test #</th>
<th>Test Name</th>
<th>Input</th>
<th>Number of cycles</th>
<th>Stroke</th>
<th>Testing conditions (V = Peak Velocity)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Thermal</td>
<td>Linear</td>
<td>1</td>
<td>± 895 mm</td>
<td>V&lt;0.05 mm/s for 5 minutes Increase velocity to 1mm/s up to completion of the displacement.</td>
</tr>
<tr>
<td>2</td>
<td>Velocity Variation</td>
<td>Sinusoidal</td>
<td>5, 5, 3, 2</td>
<td>± 300 mm</td>
<td>V=0.13 m/s, V=0.40 m/s, V=0.80 m/s, V=1.20 m/s, V=1.60 m/s</td>
</tr>
<tr>
<td>3</td>
<td>Full stroke &amp; Velocity</td>
<td>Sinusoidal or step loading</td>
<td>1</td>
<td>± 850 mm</td>
<td>V_{max}=1.6 m/s</td>
</tr>
<tr>
<td>4</td>
<td>Wear</td>
<td>Linear</td>
<td>20000</td>
<td>± 5 mm</td>
<td>V=15 mm/s Every hour change position of the piston of about 100 mm</td>
</tr>
<tr>
<td>5</td>
<td>Velocity variation</td>
<td>Sinusoidal</td>
<td>2</td>
<td>± 300 mm</td>
<td>V_{max}=1.6 m/s</td>
</tr>
</tbody>
</table>

Maximum forces appear to be very symmetric in the all range of velocity. A difference of 7% was recorded at maximum speed (1.6 m/s) only for the first cycle. The second cycle of the same test shows instead a deviation of 1.6%. The comparison among peak forces of different cycles shows a reduction of the peak force of 3.8% between fifth cycle and first cycle for test Velocity A (0.13 m/s). For the high speed tests the maximum force reduction is equal to 10.4%. 

**Figure 4** Prototype testing at SRMD Facility (UCSD)
### Table 3 Summary of Test Results

<table>
<thead>
<tr>
<th>Test name</th>
<th>Force 1st Loop (kN)</th>
<th>Force 2nd Loop (kN)</th>
<th>Force 3rd Loop (kN)</th>
<th>Force 4th Loop (kN)</th>
<th>Force 5th Loop (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thermal</td>
<td>256.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Stroke Verification 1</td>
<td>1500.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Stroke Verification 2</td>
<td>1003.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Velocity Variation A</td>
<td>2347.8</td>
<td>2315.2</td>
<td>2301.1</td>
<td>2292.2</td>
<td>2287.4</td>
</tr>
<tr>
<td>Velocity Variation B</td>
<td>2717.4</td>
<td>2730.4</td>
<td>2717.5</td>
<td>2702.3</td>
<td>2697.9</td>
</tr>
<tr>
<td>Velocity Variation C</td>
<td>3004.4</td>
<td>3021.2</td>
<td>3016.4</td>
<td>3008.1</td>
<td>3009.0</td>
</tr>
<tr>
<td>Velocity Variation D</td>
<td>3225.3</td>
<td>3215.9</td>
<td>3222.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Velocity Variation E</td>
<td>3722.9</td>
<td>3337.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Full Stroke &amp; Velocity</td>
<td>3704.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Velocity Variation F</td>
<td>3590.5</td>
<td>3310.4</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The calculated energy dissipated per cycle (EDC) indicates a reduction of 5.2% between cycles for the high speed test (Velocity Variation E).

Wear tests were completed with 10 sets of 2250 cycles, at constant velocity of 0.015 m/s and 10 mm total stroke.

![Figure 5 Damper Constitutive Law](image)

The force displacement response of the damper is reported in Figure 8 for the test at 1.2 m/s peak velocity.
Thermocouples were installed both internally and externally the damper body in order to monitor temperature rise during and after the motions. Air and nitrogen gas was used, in a cooling box, to restore the ambient temperature on the damper before a new test.

Temperature rise was recorded for each test. The maximum increase took place at the end of the test Velocity Variation B, with 40 degrees Celsius recorded from the sensor installed internally to the damper.

All the test results were deemed to be in agreement with the design specifications. It is worth to stress how the test results obtained at the SRMD facility agreed very well with the measurements performed at FIP testing laboratory and how the damper behavior resulted to provide for stable reaction in a very wide velocity range (0.002 – 1.6 m/s).

The extrapolation of the damper reaction in the range of velocity 0.13-1.6 m/s from the test performed at FIP laboratory up to 0.2 m/s differed from the measured reaction at the SRMD facility by only few percent point. This result demonstrates how much predictable is the damper behavior within the full test range.

4 FUSE RESTRAINTS

Fuse restraints shall provide for a stiff link between the deck and the substructure for the lateral service load as well as for low occurrence high-intensity windstorms. As anticipated, only at the occurrence of high intensity events they are designed fail allowing the damping system to work properly.

They are located at the main piers as well as at the transition piers. The main pier units are designed as single units equipped at their ends with spherical hinges. This configuration allows for the design rotations as well as for a correct alignment of the load along the device axis for any deck position. The element failing at the reaching of the desired design load – the so called Fuse Element - is installed in the middle of the unit, for a general configuration of the unit see Figure 9.

The units to be installed on the main piers are characterized by a failure load of 10500kN. Similarly the units installed at the transition piers, that are installed as components of the dampers, are characterized by a failure load of 3400kN. The design tolerance on the failure load of the units requires a precision of ±10%: a very strict design performance for such a high capacity units.

Furthermore, a main design complication came from the need of minimizing the internal forces of the laterally restraint deck induced by tectonic movements originated from seismic fault located under the bridge. Thus, the units located on the main piers are equipped with a system designed to allow for length adjustment. This operation is performed when a certain load level is constantly applied to the link, so a load cell monitoring the load on the unit is required. Other requirements were that the units shall not disassemble after failure of the main component and that it has to be equipped with a load cell in order to monitor the load acting on the unit. As part of the monitoring system of the bridge, this will allow to identify the moment at which re-adjustment of the deck is required.

Being their first function to withstand the every day actions (service loads), a test has been required to evaluate the fatigue life as well as any influence on the failure strength.
4.1 Testing program

The testing program was carried out on two full-scale prototypes of fuse element for each typology.

It consisted on a first test performed on one unit increasing monotonically the load up to failure, while a second test was performed on the other prototype imposing two millions of cycles at a load level equal to 10% of the design failure load and then increasing monotonically the load up to failure.

Failure test and fatigue test were carried out on different test rigs, the first one is a 8000 kN capacity test rig commonly used for bearing test while the second one is a 3000kN dynamic test rig: the same used for damper testing.

Testing results have shown that both prototypes failed within the design tolerance (±10% of the theoretical load). The very slight difference, about 1%, between the two measured failure loads can just confirm that the design fatigue load cannot be considered as effecting the ultimate capacity of the fuse restraints. In the following table, all the testing results are presented. Figure 10 shows typical graphs obtained for both the 3400kN and 10500kN units.

<table>
<thead>
<tr>
<th>Device Type</th>
<th>Failure Load Capacity (kN)</th>
<th>Tolerance Range (kN)</th>
<th>Measured Load (kN)</th>
<th>Deviation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SR340</td>
<td>3400</td>
<td>3060-3740</td>
<td>3545</td>
<td>+4.3</td>
</tr>
<tr>
<td>SR340</td>
<td>3400</td>
<td>3060-3740</td>
<td>3591</td>
<td>+5.6</td>
</tr>
<tr>
<td>SR1050</td>
<td>10500</td>
<td>9450-11550</td>
<td>11000</td>
<td>+4.8</td>
</tr>
<tr>
<td>SR1050</td>
<td>10500</td>
<td>9450-11550</td>
<td>NA</td>
<td>NA</td>
</tr>
</tbody>
</table>

Figure 11 shows the SR1050 fuse element testing configuration during the fatigue test.
5 CONCLUSIONS

Testing of full-scale hydraulic damper and fuse elements were performed in order to confirm the characteristics and the main design assumptions of the seismic dissipation system used for the Rion-Antirion Cable stayed bridge.
Tests performed on Fuse elements verified positively the design assumption showing that it is possible to achieve a very tight tolerance (±10%) on the predicted ultimate capacity even when the units are designed for a very high failure load (10500kN).

The Viscous Damper prototype demonstrated very stable behavior even when tested under dynamic conditions, requiring power dissipation higher than design parameters.

The measured energy dissipation and viscous reaction were always well within the design tolerance of ±15% of the theoretical design parameters.

Testing aimed to represent the effect of structural vibrations induced by traffic and/or of movement induced by structure thermal expansion did not produce any appreciable change in the damper behavior.

Full-scale testing of large size dissipating devices proved to be a real challenge compared to previous testing experiences as it pushed the limits of equipment available worldwide.

Experimental results on full-scale prototypes give a high degree of confidence on seismic devices as a means to protect large and important structures such as the Rion-Antirion Cable Stay bridge.

ACKNOWLEDGEMENT

The authors wish to express their sincere gratitude for the active and fruitful cooperation rendered by Mr. Gianmario Benzoni of the UCSD Laboratory, as well as the indefatigable support furnished by Mr. Renato Chiarotto of FIP Technical Department, Mr. Pierluigi Galeazzo Testing Laboratory Manager of FIP Industriale and his staff.

A remarkable thank is addressed to Mr. Agostino Marioni (ALGA) and to Mr. Philippe Salmon (Freyssinet) for their cooperation within the JV operations.

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