Seismic rocking retrofit of Christchurch's Triumphal Arch: validation of design concepts through numerical analyses

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ABSTRACT: In the last two years the region around Christchurch was subjected to several seismic events and aftershocks that damaged buildings and infrastructures. Among them there were also historically relevant heritage structures such as the Triumphal Arch, which stands on the Bridge of Remembrance in the Christchurch CBD. The structure, built in 1924, consists of a central major arch and two lateral minor arches made of lightly reinforced concrete and architectural stone. Although the Triumphal Arch did not suffer damage after 4th September 2010 event, it required strengthening works after the 22 February 2011 earthquake.

In this paper, the authors present different solutions to assess the seismic performance of for Triumphal Arch which incorporates rocking concepts. Three retrofit designs are herein shown: a pure-rocking mechanism of the arches' columns designed by SCIRT and two dissipative rocking concepts developed by the authors. The dissipative rocking solutions are based on the hybrid PRESSS technology and one aims to create a sort of frame system while the second one a coupling rocking wall system. The former uses axial external elasto-plastic dissipaters while the latter steel U-shaped flexural plates.

Numerical lumped plasticity models are developed and, by using RUAUMOKO 3D, time-history analyses are carried out as validation of the three design concepts. The three solutions are subjected to a set of ground motions which includes the Christchurch sequence. The results prove that the dissipative rocking solution reduces the displacements of the structure without inducing higher loads on foundations and limits post-earthquake damage to the replacement of the dissipaters. Accordingly it is a robust technology for seismic retrofit of heritage structures.

1 INTRODUCTION

The region in and around Christchurch was subjected to several seismic events in the last few years that devastated the entire area. The Triumphal Arch is a heritage fabric that stands on the Bridge of Remembrance, a single span arch bridge on the Avon River. The Bridge and its Arch were built in 1924 and they are located in Christchurch's city centre (43°31'59.18"S, 172°38'0.10"E) (Figure 1a).
Due to the 2011 Christchurch earthquake, the Bridge of Remembrance suffered moderate damage on the surface: this included paving damage at the approaches and widespread cracks near the base of arch. The damage increased after the 13 June 2011 aftershocks. On the other hand the Triumphal Arch did not suffer damage after 4th September 2010 event but it required strengthening works after the 22nd February 2011 earthquake. As shown in Figure 2, due to the difference of stiffness, several flexural cracks occurred at the sections which connect the upper part of the major arch to the lower part. Being

the structure really sturdy at the base, shear cracks occurred in minor arches, especially the Southern one.

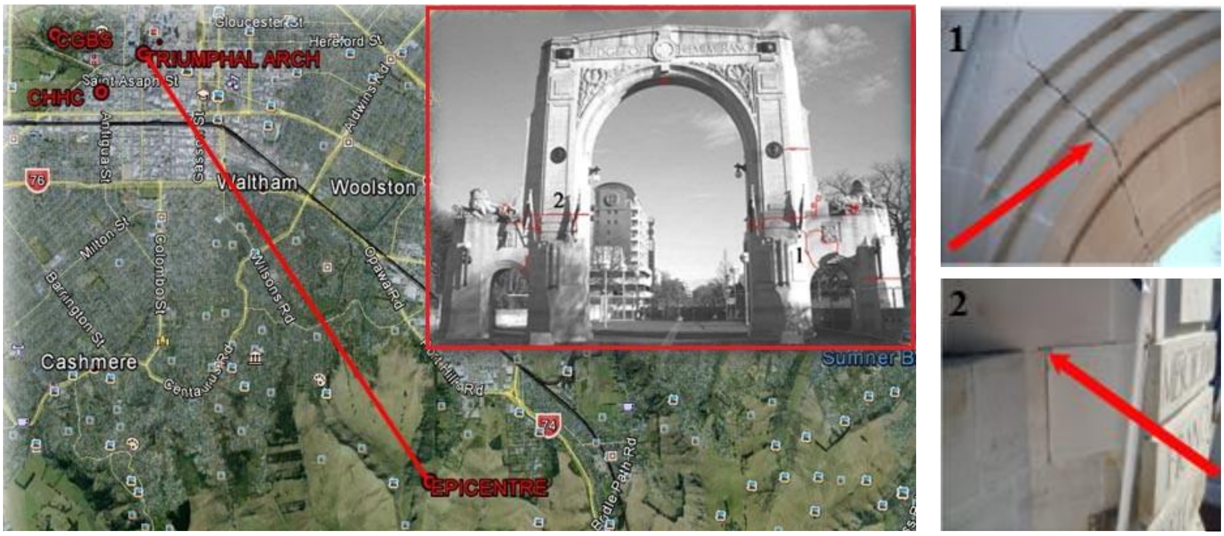


Figure 1. Crack Map for the Triumphal Arch and its location in relation with Christchurch earthquake epicenter. Zoom of cracks. Courtesy of SCIRT.

In this paper the assessment of the seismic performance of the existing structure and three different retrofit solutions are presented. The first concept is based on a pure rocking behaviour while the other two involve a hybrid rocking mechanism. To evaluate the performance of the different solutions, non-linear dynamic time-history analyses are carried out and the obtained results are herein presented.

1.1 Assessment of the Triumphal Arch original structure

The Triumphal Arch can be subdivided in a major central arch connected to two minor lateral ones. It is made by a lightly reinforced concrete hollow shell (f'_c =14 MPa and f_y =210 MPa) and it is covered by architectural stone facing (Figure 1b).

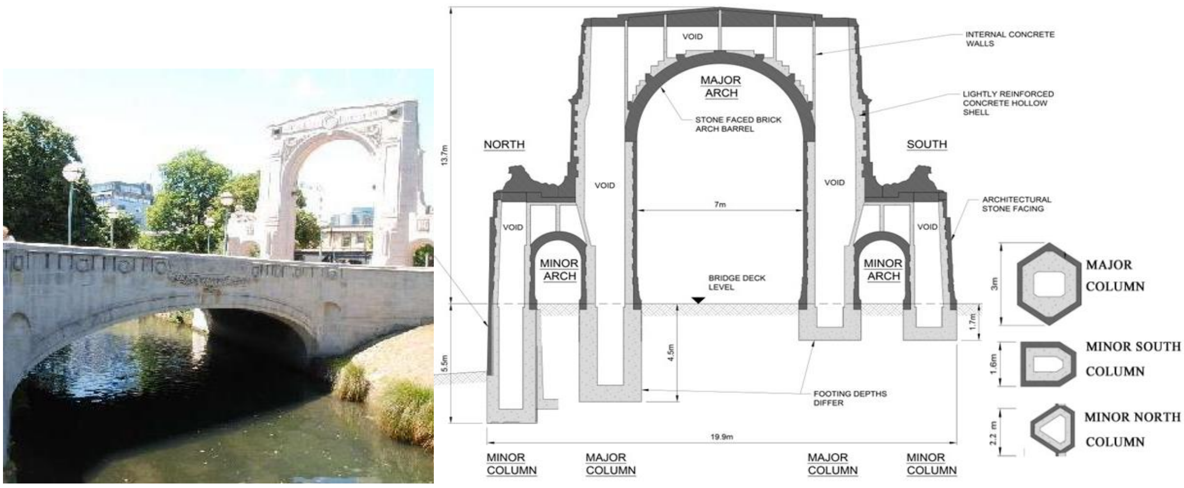


Figure 2. a) The Bridge of Remembrance and the Triumphal arch. b) Drawings of the main sections of the Triumphal Arch. Courtesy of SCIRT.

The total width and height of the major arch are respectively 19 m and 13.7. The height of the minor arches is 3.5 meters. The inner columns are characterized by a hexagonal hollow section tapered along its height and the external ones, which are significantly smaller, present slightly different irregular sections. The Triumphal Arch is founded on separate pad foundation with the same section of the lower part of the columns. Even though the upper part of the structure is almost symmetric, foundations are strongly asymmetric: the pad on which the northern column is founded is significantly deeper than the southern one. This leads to potential torsional effects. The structure behaves basically as a frame in the in-plane direction and as a cantilever in the out of plane direction so it was represented as a three portal frame (Figure 3a). Geometrical section data were collected from the original drawings supplied by SCIRT. On this basis, section analyses were carried out and the obtained moment-curvature graphs were adopted to set section parameters. As expected, the elements turned out to have a brittle behavior (Figure 3b). Taking into account these considerations, a modified Takeda hysteresis with strength degradation was chosen and the elements were modeled using a Giberson one component member. The type of foundation and soil characteristic allowed considering the columns fully fixed at the base. The so modeled structure has respectively a natural period of 0.149s and of 0.13s in the out-of-plane and in-plane direction.

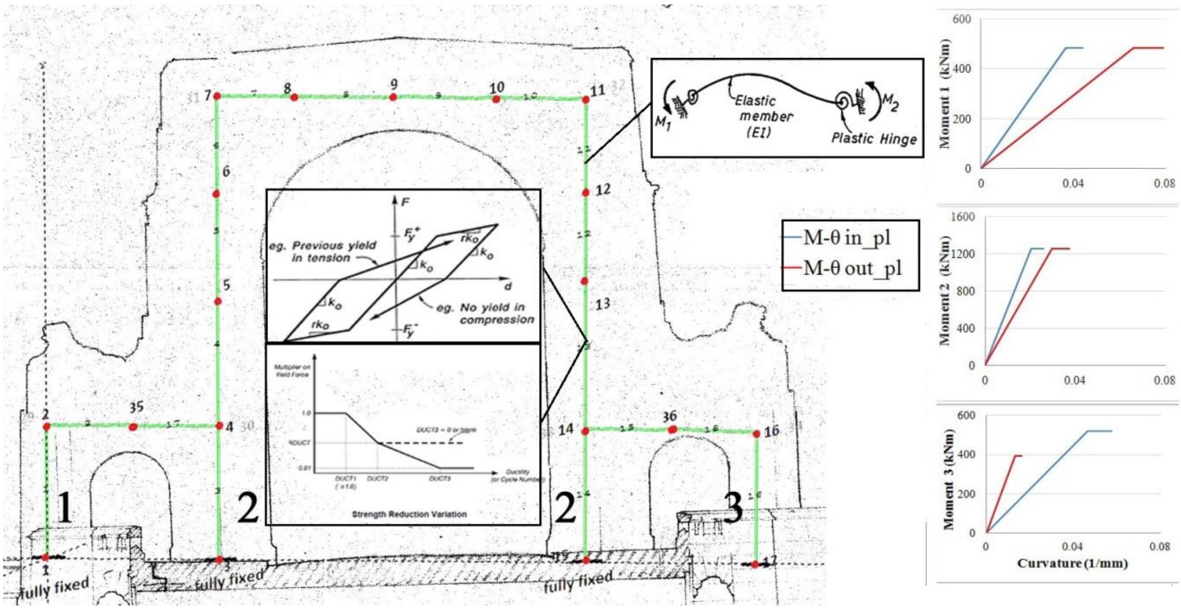


Figure 3. a) Ruaumoko model of the original structure. b) Moment-Curvature graphs of the columns

Non-linear time history analyses were also carried out. Analyses are performed with the software Ruaumoko 3D (Carr, 2008) using existing earthquake records from September 2010 and February 2011. Data records from CHHC (43°32'15"S, 172°37'38"E) and CBGS (43°31'52"S, 172°37'11"E) Strong Motion Accelerographs are used. The force-displacement graphs obtained applying the seismic load (Darfield and Christchurch earthquakes) in the in-plane direction are plotted in figure 4 together with the NZS1170 force-displacement elastic response spectra. For sake of brevity, the out of plane performance of the structure is not herein reported since the behaviour is similar to the in-plane one.

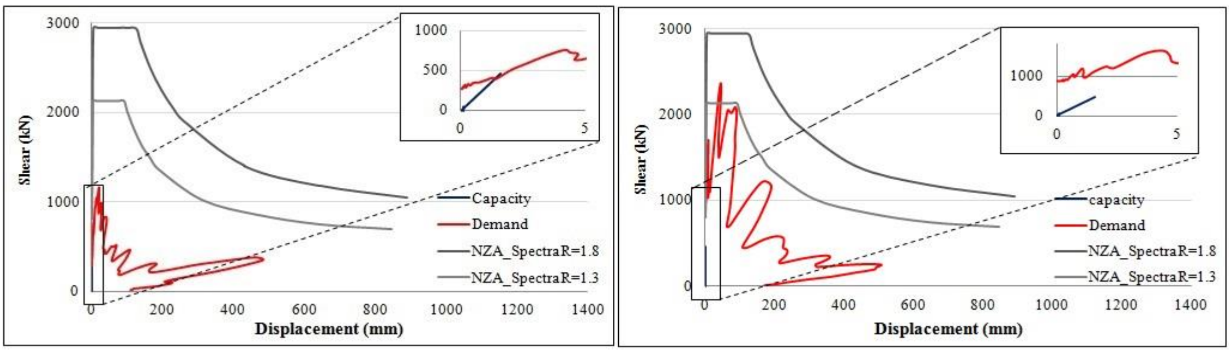


Figure 3. Spectra-displacement graph of the structure's capacity compared with NZS1170 response spectra, Darfield (left) and Christchurch (right eqs).

The model predicts the behaviour of the structure quite accurately especially when Darfiled's recordings are applied as seismic loads. As highlighted in Figure 4, when the structure is subjected to Christchurch earthquake accelerations, the collapse of the arch occurs. As expected, the obtained results reflect the difference of stiffness between the upper and the lower structure. In fact, the section of the central arch which connects these two parts is the first one that reaches the "plastic range". It can be point out that yielding moment is reached in the sections where cracking occurred in the existing arch. The out of plane drift is higher (about 0.03%) than the in-plane one (about 0.01%); in fact the structure is similar to a pendulum in the z direction (out of plane) while it behaves as a frame in the x direction (plane). Low drift values are obviously expected due to the high stiffness of the structure and its brittle failure. A brief summary of the comparison between capacity and demand of the original structure under the considered seismic loads is made in the following table.

Table 1. Demand (Darfield and Christchurch Earthquake) and capacity of the original structure in term of base shear and top displacement.

		V (kN)	**V (kN)**	**Drift (%)**	**Drift (%)**
		In-plane	Out-of-plane	In-plane	Out-of-plane
Capacity		551	425	0.033	0.0146
Darfield Eqs	CBGS	333.83	406.99	0.0255	0.0168
	CHHC	215.79	286.95	0.0255	0.00956
22 February Eqs	CBGS	671.88	1213.19	0.0264	0.0404
	CHHC	614.24	638.19	0.0144	0.0219

2 ROCKING AND DISSAPATIVE CONTROLLED ROCKING

Rocking motion is a common phenomenon that takes place in structures during a seismic event and was firstly investigated by Housner (1963). It is a complex and highly non-linear phenomenon and it is actually considered a viable isolation solution at the base of the structure. Indeed, the resulting acceleration and internal forces are significantly reduced because the natural period of the structure is increased (as shown Figure 5, left). More importantly, if foundation edges are properly design, this technology reduces to minimal the post-earthquake cost of repair (Kawashima, 2005). In light of growing demand for enhanced seismic performance of structures, which minimize post-earthquake damage, pure rocking motion was successively "controlled" by adding additional restoring force and made more dissipative by adding fuses at the connection with the foundation. The concept of dissipative controlled rocking (DCR) found its first use in the U.S. PRESSS program (hybrid connection), Priestley,1997, Priestley et al. 1999. The concept of combining self-centering action and energy dissipation is material independent and was then extended to steel frame structures (Christopoulus et al. 2002) and bridge piers (Hews and Priestley,1997; Palermo et al 2005) and timber buildings (Palermo, 2005). The hybrid connections are generally composed by unbounded post-tensioned tendons (elastic behavior) and dissipaters such as internal de-bounded bars, axial dissipaters (Sarti et al., 2013) and UFPs. Alternatively advanced engineered materials, such as Shape Memory Alloy (SMA) can reproduce a similar behavior (Des Roches, 2007) to a DCR or hybrid connection. Such solution creates a system that exhibits a so called "flag-shape" behavior which has the advantages of providing dissipative capacity as well as re-centering capacity (reduction of the residual deformation and closure of existing cracks (Christopoulus et al. 2003)). Differently from a structure that purely rocks on its foundation, the dissipative controlled rocking solution does not drastically shift the period of the structure but provides ductility capacity through the dissipative fuses. The ductility factor μ of DCR or Hybrid PRESS systems can easily reach a value around 3, 4 at ULS (Ultimate Limite State), as shown in Figure 5. The main driver of the overall ductility capacity of the system are the dissipative fuses, therefore different ductility factors should be adopted based on the performance based criteria of the device. Both rocking and dissipative controlled rocking prevents the creation of plastic hinges that characterizes the behavior of cast-in situ R.C structure and therefore can be both classified as “Low or No post-earthquake damage” structural techniques. For iconic structures such as Bridge of Remembrance, Triumphal Arch is essential that the structure suffer minimal damage even after a strong seismic event in order to avoid irrecoverable loss of heritage fabric. Both the appearance and the retention of the original material are considered essential aims. Loss modelling analyses (Marriott, 2009) proved that DCR are viable cost-effective solutions for seismic design and retrofit of structures.

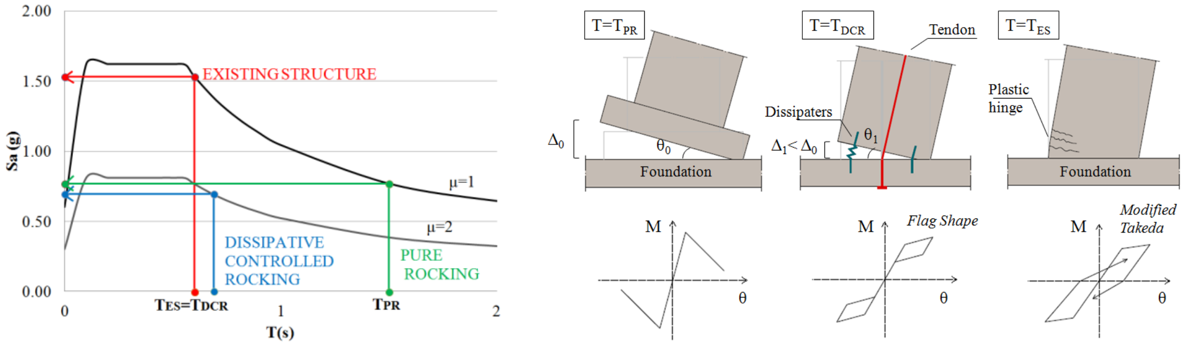


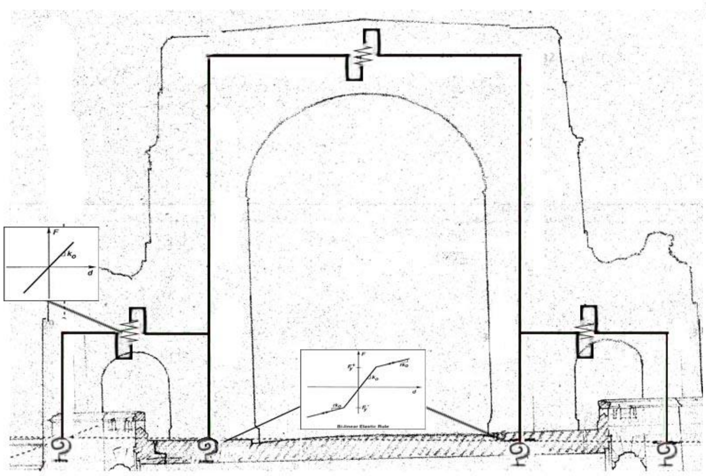
Figure 4. Comparison between the behavior of the existing structure, a pure rocking and a dissipative rocking one: a) spectra response accelerogram; b) hysteresis rules and sketches of displaced configuration

Historical buildings, as the Triumphal Arch, are generally characterized by relatively low natural vibration period. Hence, if they are located in seismic areas, rocking retrofit design are often carried out in order to reduce the acceleration and internal forces during a seismic event. The key advantages of using this solution for historical buildings is that it provides protection of the original structure, avoiding irrecoverable damaging to the heritage architecture and more importantly doesn’t impact on strengthening of existing foundations.

2.1 Pure rocking solution

2.1.1 *Concept*

One possible solution to prevent further damage to the fabric and the collapse of the structure is to provide adequate rocking capacity such that the natural period of the structure is increased and the internal loads are significantly reduced compared to the existing arch. This solution has been adopted by Stronger Chirstchurch Infrastructure Rebuild Team (SCIRT) designers. The retrofit strategy consists of strengthening remedial works of the original foundation and filling the internal voids of the columns with new reinforced concrete and with fabricated steel boxes (SCIRT). This prevents the frame to damage during the rocking motion. Rocking collars at the base of the columns are added in order to allow for the same rocking movement in the four columns, while vertical sliding joints above the crown of each of the three arches accommodate displacement uplifts induced by the rocking of the collars.



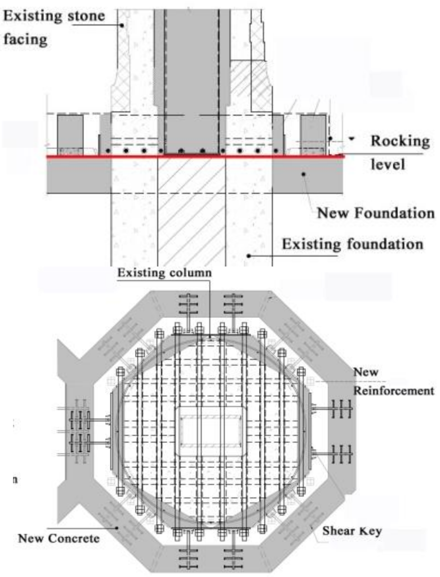
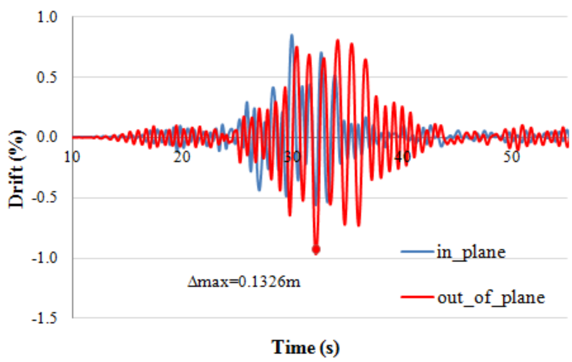


Figure 5. Sketch of the pure rocking model. Section and elevation of the rocking collar (Courtesy of SCIRT).

Horizontal unbounded post-tensioning tendons passing through the vertical sliding joint interface allow the rocking motion of the whole structure and were also inserted and design to resist lateral torsional movements.

2.1.2 *Numerical model and non –linear time history analysis*

The structural model is similar to the one in paragraph 1.1 although some key changes were introduced at the base of the frame. First of all, taking into consideration the strengthening solutions new section parameters were defined for the frame elements. Frame elements were designed to behave elastically; consequently the gross un-cracked section properties were used. Secondly, the hinge joints at the top of the arches were modelled by releasing the vertical displacement degree of freedom and slaving the other displacements, while previously all degree of freedom were slaved. In addition, spring elements were introduced in the arches to simulate the unbounded post-tensioning; the stiffness was function of the number of cables used and the material properties. Furthermore, different external constraints were introduced: translations and torsional rotation were blocked while bending rotations were ruled by a zero length spring at the base of each column. The properties of the springs were set on the basis of moment-rotation curves determined with section analyses at the level of the rocking plane. The total moment of yielding in correspondence of the rocking section was 5000 kNm. Since it relies on the gravity load, the stiffness of the springs of the major columns was higher than the one of the minor columns despite the fact that the collars have the same geometry. In order to guarantee the continuity of displacements, the yielding rotation was set to be the same for all springs.



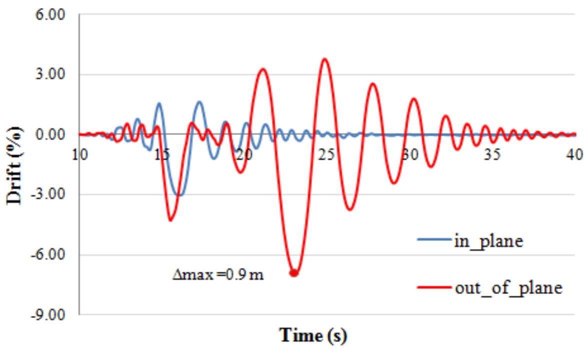


Figure 6. Top drift of the pure rocking structure: Darfield (left) and Christchurch earthquake (right) (CBGS Recordings).

Table 2. Performance (Darfield and Christchurch Eqs) of the pure rocking structure: base shear and top drift.

		V (kN)	V (kN)	Drift (%)	Drift (%)
		In-plane	Out-of-plane	In-plane	Out-of-plane
Darfield Eqs	CBGS	898	716.45	0.81	1.02
	CHHC	774.3	716.45	0.803	1.1678
22 February Eqs	CBGS	800.3	921	3.01	5.45
	CHHC	881.9	1041	2.45	5

The time-history analyses were carried out applying the same recordings used for the original structure and the obtained results are plotted in Figure 7. Being the structure less stiff in the out of plane direction, the correspondent drifts are sensible higher than the in-plane ones. It has to be pointed out that the seismic loads induce a higher shear in the rocking structure than in the existing one. This fact is due to the significant increment of mass related with the strengthening works. As a matter of fact, the addition of reinforcement and concrete was unavoidable since the existing structure couldn't withstand even the smaller amount of shear demand correlated with its smaller mass. It worth pointing out that, thanks to the significant width of the rocking collars, the P-Δ effects were negligible (6% of the total base moment).

2.2 Seismic dissipative rocking retrofit solution

2.2.1 *Concept*

Two dissipative rocking solutions were also developed by the University of Canterbury researchers. Both solutions are based on the hybrid PRESSS technology: the first solution, similarly to SCIRT's design aims to create a coupling rocking wall system; in the second solution of the structure performs as a frame system. As previously discussed, DCR doesn't drastically shift the natural period of vibration as in the pure rocking design but introduces ductility to the system leading to a reduction of the seismic loads. For sake of brevity, only the results obtained for the first solution are here reported.

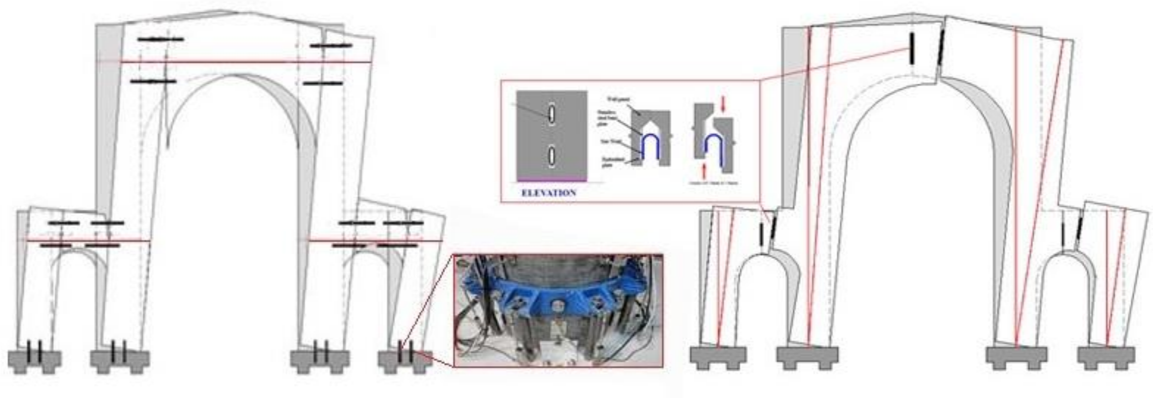


Figure 7. Dissipative controlled rocking concept: frame behavior (left) and coupled walls behavior (right).

The first solution is similar to coupled wall systems and aims to reduce the damage allowing the rocking motion on foundations and dissipate energy through the UFPs, which are activated by the relative movements of the half-arches. The vertical post-tensioning members clamp columns to their foundations as well as increase the lateral resistance of the system when the rotation mechanism takes place, providing a self-centering action. The second solution creates a sort of rocking frame with dissipaters located at the base of the four columns, while at beam-column connections horizontal post-tensioning tendons pass through the arches (figure 6). External dissipaters may be used to allow simpler post-earthquake repair. New grouted concrete and stone facing in order to maintain the aesthetic of the heritage fabric, similarly to SCIRT's design can hide the collars and dissipaters.

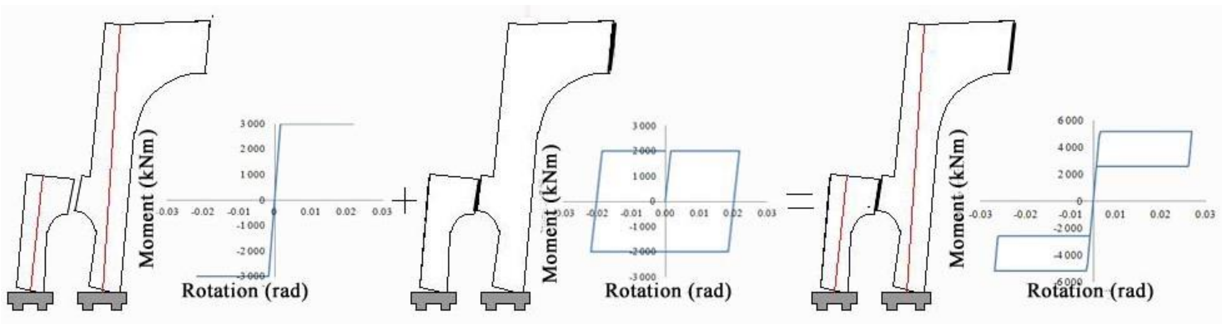


Figure 9. Flag shape hysteresis of the rotational spring.

2.2.2 *Numerical model and non –linear time history analyses*

The analyses of the coupled wall system, i.e. first solution, are carry out by using a model based on frame elements. Spring elements representing the other components of the system (dissipaters, tendons and UFPs) are the connection between the elements. Similarly to the pure rocking solution, the arch is modeled as a three portal frame with sliding joint in the mid sections; similar section parameters of the pure rocking structure were used. The frame elements behave elastically while the dissipative controlled rocking is simply modeled with rotational springs at the base with flag shape hysteresis. This simplified approach should produce different results of more refined numerical modeling techniques proposed by (Spiethz, 2004, Palermo 2005). The flag-shape hysteretic behavior is given by the combination of a non-linear elastic (tendons) and the elasto-plastic hysteresis (UFPs) (Figure 9).

Same set of analyses have been repeated using the same records as in paragraph 2.12. Base shear and top displacement calculated through time history analyses are listed in Table 3. On the contrary of what was obtained for the previous structures, when Darfield recordings are applied the in-plane drift is higher than the out of plane one. This might be expected since the accelerogram response spectra of the seismic event shows that, for the periods of the DRC structure, the principal direction of shaking was the in-plane one.

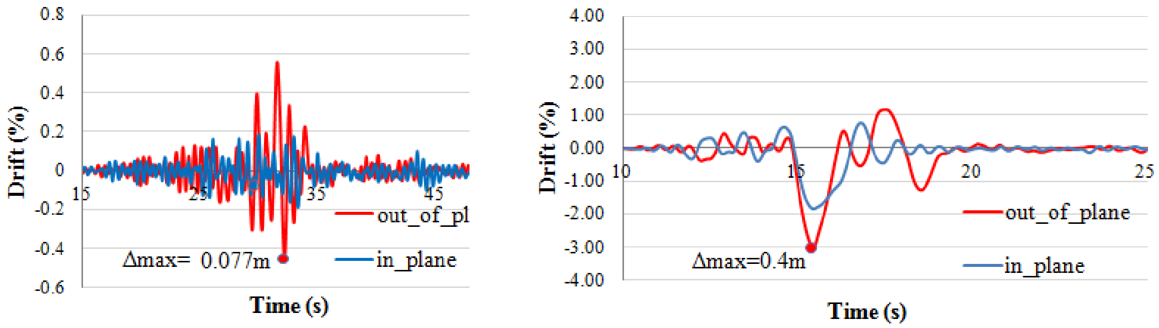


Figure 8. Top drift of the DCR structure: Darfield (left) and Christchurch earthquake (right) (CBGS Recordings).

Table 3. Performance (Darfield and Christchurch Earthquake) of the DCR structure in term of base shear and top drift.

		V(kN)	**V (kN)**	**Drift (%)**	**Drift (%)**
		In-plane	Out-of-plane	In-plane	Out-of-plane
Darfield Eqs	CBGS	721	677	0.41	0.292
	CHHC	743	580	0.657	0.314
22 February Eqs	CBGS	1042	940	1.85	3
	CHHC	847	870	1.02	3.01

3 CONCLUSIONS

The 22nd February 2011 earthquake was an exceptional event where the Triumphal Arch, as most of Christchurch bridges did not suffer significantly damage but it required strengthening. As many other historical buildings, it is characterized by relatively low natural vibration period and a brittle behavior. Strengthening by using steel jacketing or FRP (fiber reinforced polymer) would have improved strength and ductility of the system but causing excessive post-earthquake disruption. Pure rocking solutions or dissipative controlled rocking seem to be viable solutions. Firstly because they both limit the damage to possible concrete crushing at the edges of the rocking foundation and secondly because the base shear forces are equal or less then the existing structure. This allows to drastically limiting the cost of retrofit to the structure only without strengthening the foundation.

Both solutions if properly detailed can preserve the architectural heritage. On the basis of the results obtained with time history analyses carried out with the set of ground motions of the Christchurch sequence, the original structure was too brittle to withstand Christchurch earthquake. On the other hand, both rocking solutions showed that base shear forces were limited despite higher displacement demands compared to the existing structure. Dissipative Controlled Rocking has more robustness then Pure Rocking, since it introduces more redundancy in the rocking connection and supplemental damping. Moreover, the displacement demand is lower then Pure Rocking solution and this aspect can be critical for protecting/preventing damage of the fabric and non structural parts (parapets, stones etc.)

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