Application of hybrid concept for an improved seismic ductile design of bridges

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ABSTRACT: Following the research developments in the seismic design of precast frame and shear wall systems based on innovative jointed ductile connections, similar innovative alternative solutions can be proposed for concrete piers for an improved seismic performance when compared with traditional monolithic solutions. In particular, the possible extension to bridge piers and system of the concept of a peculiar jointed ductile connection, the hybrid system, where self-centring properties (unbonded post-tensioned cables) are adequately combined with additional energy dissipation (longitudinal mild steel bars or energy dissipation devices), is herein considered and investigated. After brief introduction on the development of the hybrid systems, a description of the peculiarities of their cyclic behaviour is carried out, considering the most significant parameters governing the response. A critical comparison of the seismic response, in the transverse direction, of hybrid and monolithic connections at both local (single bridge piers) and global level (bridge systems) is carried out through inelastic dynamic time history analyses using lumped plasticity models.

1 INTRODUCTION

During the past two decades, earthquakes have caused damage (often sever, beyond reparability condition) to a considerable number of bridges even when designed according to major code standards. Traditional seismic ductile design philosophy of concrete bridges implies the inelastic response of the structure to occur within plastic hinge regions at the bottom and/or top of the pier elements. According to a performance-based seismic design approach, different levels of structural damage and, consequently, repairing costs are thus expected and, depending on the seismic intensity, typically accepted as unavoidable result of the inelastic behaviour. As a consequence, a significant effort has been made to develop innovative design approaches and structural systems able to limit the damage of the structure after a seismic event.

The development of alternative solutions for precast concrete buildings based on jointed ductile connections, under the U.S.-PRESSS Program (PREcast Seismic Structural System Structural), coordinated by the University of California, San Diego, (Priestley et al. 1999) has introduced an innovative concept in the seismic design of lateral load resisting systems: alternatively to the emulation of cast-in-place approach, pure precast elements are assembled through post-tensioning techniques, with the inelastic demand being accommodated within the connection itself with no damage in the structural elements.
A particularly efficient solution was given by the hybrid system, based on an adequate combination of self-centring properties (provided by unbonded post-tensioned tendons/cables plus axial load) and of energy dissipation capacity (mild steel or additional dissipation devices).

In this contribution the concept of hybrid or "controlled rocking" connections is proposed to be extended to bridge piers and systems (Fig. 1) and numerically investigated in order to verify its efficiency and potentiality. Comparisons with the performance of traditional cast-in-situ reinforced concrete solutions will be critically discussed both at local (bridge piers) and global level (bridge systems) through dynamic time history analyses adopting a lumped plasticity modelling.

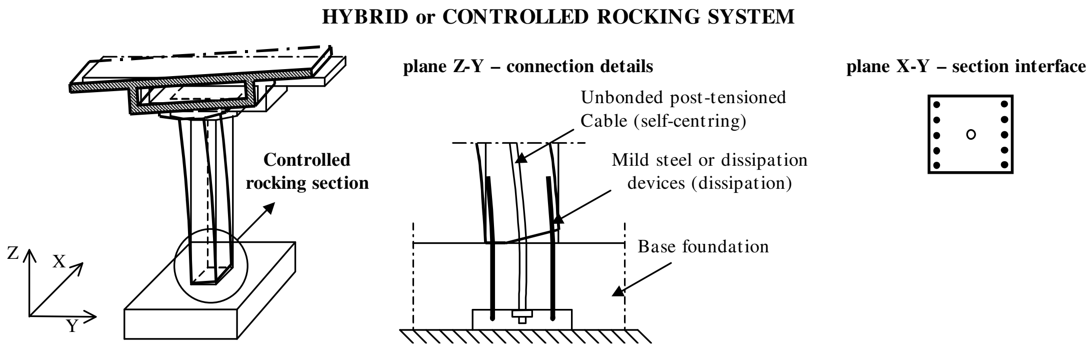

Figure 1. Hybrid or controlled rocking bridge system.

2 DEVELOPMENT OF THE HYBRID SYSTEM CONCEPT FOR BRIDGE PIERS

2.1 *Brief Literature Review*

Few contributions available in literature have been focused on the seismic response of bridge piers implementing similar innovative connections in order to improve the global seismic response of traditional bridge systems. Moreover, due to the current limit in structural testing facilities to accommodate full-scale tests on complete bridge systems, most of the experimental investigations are exclusively limited to the seismic behaviour of the single pier member. The first experimental investigations to bridge piers with pure rocking on its foundation have been carried out by (McManus 1980); an experimental application of non-pure rocking response of bridge piers (shake table tests on bridge piers, with unbonded post-tensioned cables combined with viscous dissipation devices) were successively proposed by (Mander & Chen 1997) at the University of Buffalo. Analytical-experimental investigations on precast segmented piers with unbonded post-tension cables, where no supplemental damping or additional energy dissipation devices were added to the pier system, have been performed at University of California, San Diego (Hewes & Priestley 2001). Following the extensive damages, often beyond the reparability limit, observed on several bridge piers after the Kobe earthquake (1995), several experimental studies have been promoted in Japan on prestressed or partially prestressed bridge solutions in order to improve the structural response under a seismic event and limit the damage level as well as the permanent displacements, as reported in (Kawashima 2002).

2.2 *Seismic Design of Hybrid Bridge Piers: the "Flag-shape" Hysteresis Behaviour*

The concept of hybrid or "controlled rocking" system, as originally proposed for precast concrete building, is herein proposed to bridge piers and systems for an improved seismic performance. By adequately selecting the ratio of self-centring (non linear elastic behaviour provided by the unbonded tendons and axial loads) and energy dissipation moment contributions (elasto-plastic or similar behaviour given by mild steel or energy dissipation devices), defined by the parameter λ, a peculiar hysteresis behaviour, typically referred to as "flag-shape" (Fig. 2a) can be controlled, guaranteeing limited maximum displacement/drift and no residual displacement/drift.

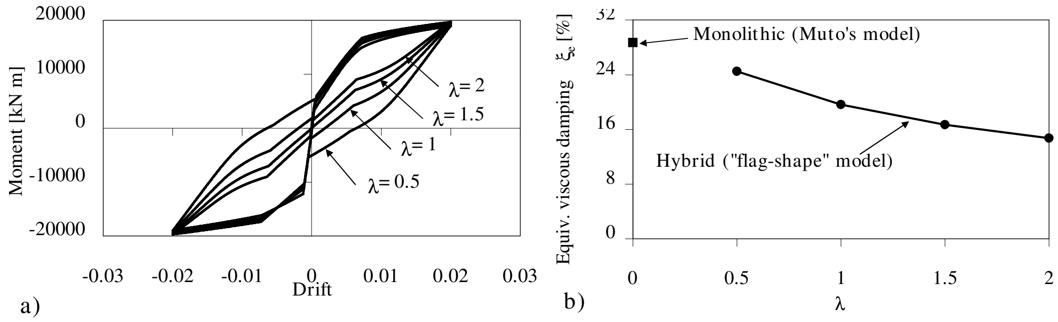

Figure 2. a) Moment vs. drift varying λ; b) equivalent viscous damping vs. λ.

The critical role of residual (permanent) deformations as additional and complementary indicator of damage within a performance-based design approach, has been recently emphasized in literature (MacRae & Kawashima 1997), (Pampanin et al., 2002). As shown in Fig. 2b, a balanced design value of λ can be suggested in the range between 1 and 1.5; higher values of λ would in fact lead to a reduction of hysteretic dissipation ($\xi_e \approx$15-12%), with consequent increase of the expected maximum displacements, as shown by the time history analyses presented in the next paragraphs. Lower values would, on the other end, not guarantee a full re-centering capability.

3 IMPROVED SEISMIC RESPONSE OF HYBRID BRIDGE PIERS: COMPARISON WITH MONOLITHIC SOLUTIONS

Extended numerical investigations based on cyclic push-pull and non-linear time-history analyses on the behaviour of a single pier with controlled rocking including a critical comparison with the seismic response of a monolithic solution have been recently presented by (Palermo 2004). A brief summary is herein reported.

3.1 *Numerical Modelling*

A lumped plasticity approach has been used to model the behaviour of piers with either hybrid connections and monolithic solutions. Elastic beam elements were used to model the bridge piers, while inelastic rotational springs with appropriate hysteresis loops were used at the critical interface sections to represent either the opening and closing of the gap during the rocking motion in the hybrid systems, as well as the formation of plastic hinges in the monolithic connections.
According to the modelling approach proposed by Pampanin et al. (2001) for hybrid precast frame and wall connection/systems and successively extended by Palermo (2004) for bridge piers, the idealized "flag-shape" hysteresis behaviour (i.e. moment-rotation), can be obtained by the combination of two rotational springs in parallel. For the following analyses, a trilinear Non Linear Elastic curve (comprising of decompression, yielding and failure points) has been adopted for the rotational spring representing the self-centring contribution, while a Ramberg-Osgood hysteresis rule has been chosen for the rotational spring representing the dissipative contribution, i.e. mild steel reinforcement passing through the critical section or external tension-compression yielding dissipation devices. For the monolithic connections, the inelastic behaviour of the plastic hinges can be modelled by a rotational spring with a modified-Takeda hysteresis rule or similar stiffness degrading rules with trilinear monotonic behaviour (Muto hysteresis rule).

3.2 *Role of λ parameter on Maximum and Residual Displacement Demand*

Non-linear time-history analyses on single bridge piers (with geometric and mechanical characteristics shown in Fig. 6a, 6b) were carried out using the finite element code RUAUMOKO (Carr, 2004). An ensemble of 10 Californian earthquake records (Table 1), scaled to match the design response spectrum provided by the International Building Code for a soil class C was used. This level is defined as two thirds of the Maximum Considered Event (MCE) spectrum (10% probability of exceedence in 50 years) for accelerations of S_s=1.5 g for the short period range and S_1=0.10 g for a period of one second and also corresponds to the Eurocode 8 design spectra for soil class B, S=1.2 and mean PGA of 0.36g. Scaled values of peak ground acceleration (PGA) are reported in Table 1.

In order to compare the performance of the two systems, the critical sections A-A, (Fig. 3) at the base-to-foundation interface were assigned similar moment-rotation monotonic behaviour (envelope curves) after adopting a Direct Displacement Based Design (DDBD) procedure (Priestley, 1998) for the monolithic case.

In Figure 4a, the maximum drifts (both positive and negative directions, mean values over the ensemble of 10 records) are reported as a function of the λ parameter (ratio of re-centring and dissipating design moment contributions). For increasing values of λ, increased level of maximum drifts are observed due to the reduced energy dissipation capacity of the hybrid connection. In particular, for a range of λ between 1 and 1.5, the increment in drift demand, when compared to the monolithic solution, is around 10%, reaching 17-18% for values of λ close to 2. In Figure 4b the residual drifts (maximum values in positive and negative directions, i.e. mean values over the ensemble of 10 records) are reported. While for the hybrid connections they can be assumed negligible, for the monolithic solution drift values of 0.7% can be reached (around 30% of maximum drift). As a consequence, higher damage and hence higher repairing costs should be expected for the monolithic solution.

Table 1. Characteristics of considered seismic records (Pampanin et al., 2002).

	Earthquake event	Year	M_w	Station	Soil type	Scaling factor	Scaled PGA [g]
EQ1	Loma Prieta	1989	6.9	Hollister Diff. Array	D	1.3	0.363
EQ2	Loma Prieta	1989	6.9	Gilroy Array #7	D	2.0	0.452
EQ3	Landers	1992	7.3	Desert Hot Springs	C	2.7	0.416
EQ4	Landers	1992	7.3	Yermo Fire Station	D	2.2	0.334
EQ5	Cape Mendocino	1992	7.1	Rio Dell Overpass-FF	C	1.2	0.462
EQ6	Superstition Hills	1987	6.7	Plaster City	D	2.2	0.409
EQ7	Northridge	1994	6.7	Canoga Park-Topanga Can	D	1.2	0.427
EQ8	Northridge	1994	6.7	Beverly Hills 14145 Mulhol	C	0.9	0.374
EQ9	Northridge	1994	6.7	N Holliwoord – Coldwater Can	C	1.7	0.461
EQ10	Northridge	1994	6.7	Sunland-Mt. Gleason Ave	C	2.2	0.345

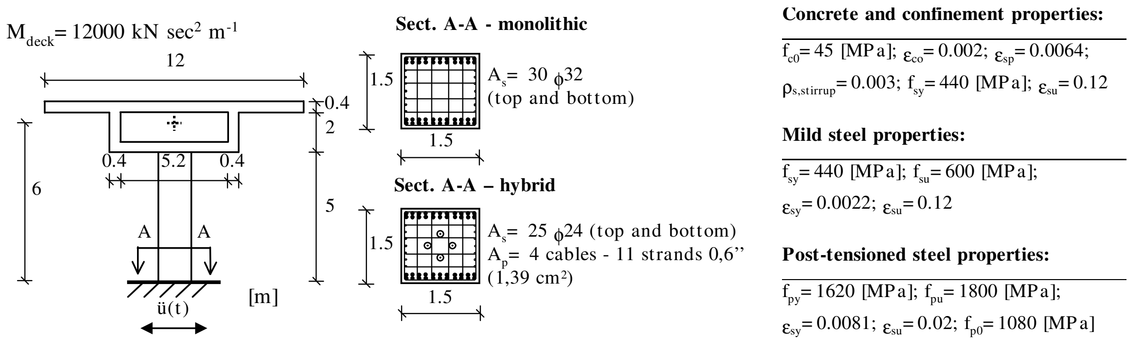

Figure 3. Bridge piers: geometric and mechanical data of the hybrid and monolithic solutions.

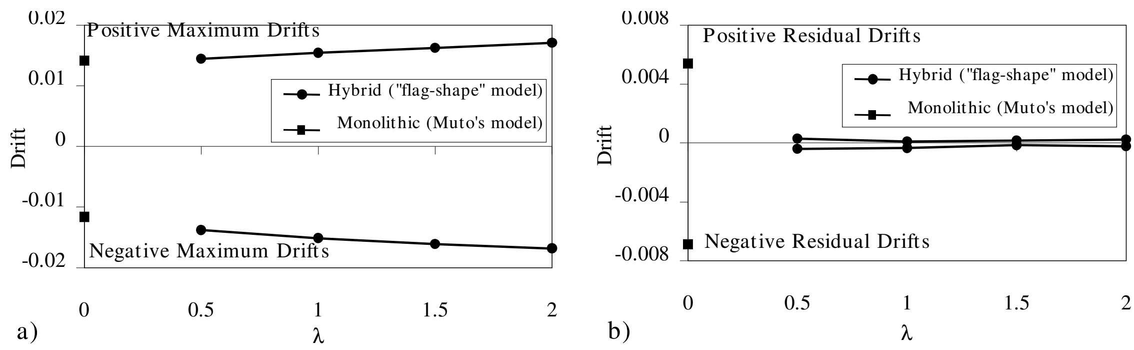

Figure 4. a) Mean maximum drift vs. λ; b) mean residual drift vs. λ.

In Fig. 5 the drift time-histories of both connections under EQ1 record (Loma Prieta, 1989) are presented. No major differences can be observed in terms of maximum drift (with the hybrid connection showing 10% higher values than the monolithic connection) while, again, substantial differences have to be highlighted in terms of residual drift, being negligible for the hybrid solution and in the order of 30% the maximum drift for the monolithic solution. The different performance is emphasized by the moment–rotation curves: in addition to an inherent self-centring capacity, the hybrid connection shows, when compared to the monolithic solution, a more symmetric behaviour in terms of rotation at the pier/foundation interface (i.e. lower difference between maximum rotations in the positive and negative directions). A summary of the numerical results for each earthquake event is reported in Table 2. with mean and standard deviation values of maximum drift (for both monolithic and hybrid connections with different values of λ parameter), residual drift and ratio of residual/maximum parameters (for monolithic connection).

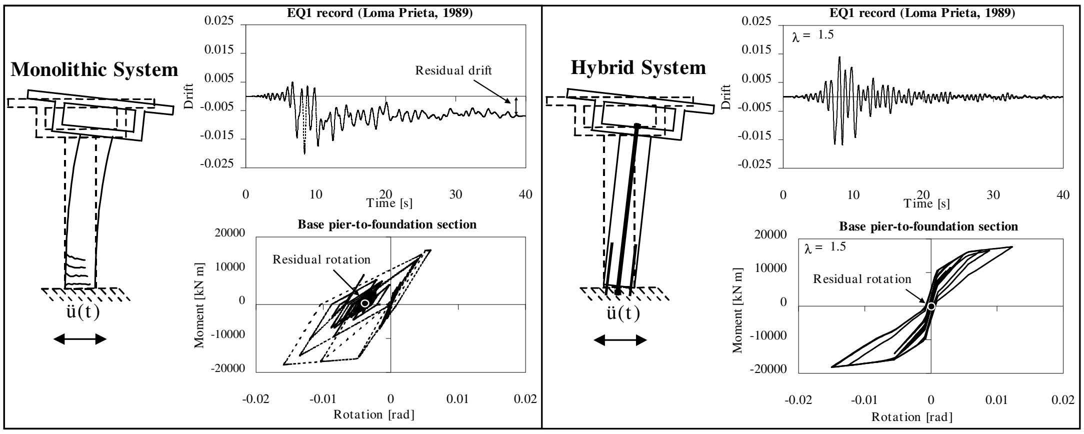

Figure 5. Seismic response of bridge piers with hybrid and monolithic connection (EQ1, Loma Prieta, 1989).

Table 2. Absolute maximum and residual drifts.

	Monolithic			Hybrid			
Earthquake event	$Drift_{max}$ [%]	$Drift_{res}$ [%]	$\Delta_{res}/\Delta_{max}$	$Drift_{max}$ [%] λ=0.5	$Drift_{max}$ [%] λ=1.0	$Drift_{max}$ [%] λ=1.5	$Drift_{max}$ [%] λ=2.0
EQ1	2.0080	0.6886	0.3429	1.6277	1.6508	1.6758	1.7170
EQ2	0.8571	0.0818	0.0955	0.7989	0.8194	0.8392	0.8499
EQ3	2.3005	0.5079	0.2208	2.4215	2.6333	2.7537	2.8650
EQ4	1.4964	0.1468	0.0981	1.7553	1.9927	2.2085	2.3440
EQ5	1.8527	0.3954	0.2134	2.1682	2.2152	2.2410	2.2427
EQ6	1.2967	0.3591	0.2769	1.0321	1.0974	1.1326	1.1521
EQ7	1.8883	0.5929	0.3140	1.7397	1.6020	1.5375	1.4866
EQ8	1.9318	0.0311	0.0161	2.3458	2.6138	2.8755	3.0560
EQ9	1.3497	0.3461	0.2564	1.1256	1.2799	1.3724	1.4890
EQ10	1.8905	0.5389	0.2850	1.5217	1.7063	1.8163	1.8918
MEAN	**1.6515**	**0.3333**	**0.1974**	**1.6565**	**1.7733**	**1.8641**	**1.9308**
STDV	**0.4367**	**0.2044**	**0.1031**	**0.5878**	**0.6455**	**0.7081**	**0.7563**

4 SEISMIC RESPONSE OF HYBRID BRIDGE SYSTEMS: COMPARISON WITH MONOLITHIC SOLUTIONS

In this paragraph the seismic transverse response of three different bridges (A3, B3, C3), whose geometry is reported in Figure 6, considering the effects of structural irregularity given by different distribution of pier heights, is investigated though dynamic non-linear time history analyses.
A three-dimensional lumped plasticity model based on the concepts previously described for the 2-D response is adopted. A Non Linear Elastic rule with tri-linear envelope curve and a Ramberg-Osgood hysteresis rule are combined in parallel to define the flag-shape behaviour of the hybrid systems, while a Muto hysteresis rule with a tri-linear monotonic behaviour is used to represent the plastic hinge behaviour in the monolithic connections. Deck and pier elements are assumed to have an elastic behaviour. A constant distribution of λ parameter along the bridge piers, with an average value of 1.5, is assumed. The same ensemble of 10 earthquake records, reported in Table 1, has been adopted for the analyses. All bridges have been designed according to the DDBD Approach (Kowalsky 2002), with a target drift θ_d in the central pier of 2%. The dynamic global properties (equivalent SDOF system) are reported in Table 3.

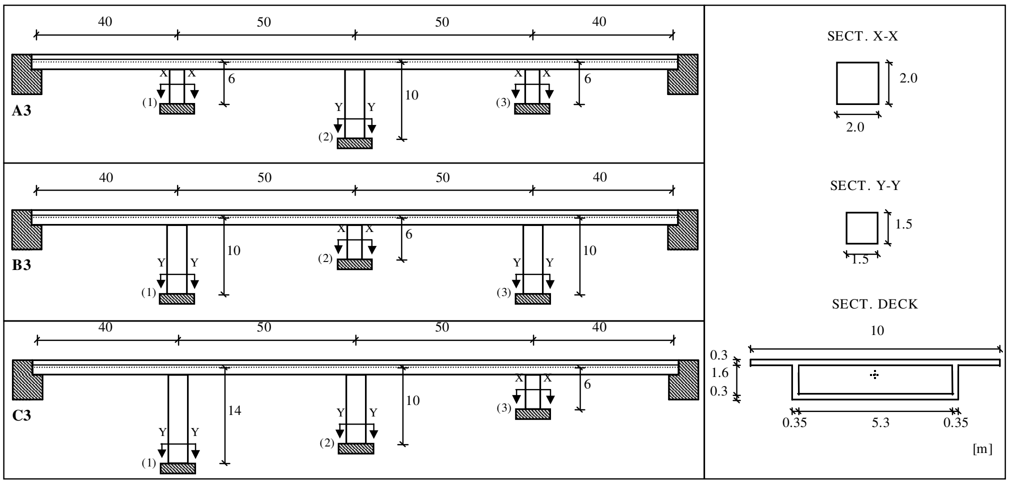

Figure 6. Geometric characteristics of bridge systems A3, B3, C3.

Table 3. Dynamic global properties of bridges A3, B3, C3 with monolithic connections (equivalent SDOF system).

Case Studies	$T_{equiv.}$ [s]	$\xi_{equiv.}$ [%]	V_{equiv} (kN)	M_i [kNm]
Bridge A3	1.77	17.60	4765	6525
Bridge B3	0.95	13.16	9875	22727
Bridge C3	1.72	15.16	5247	9515

In a first stage, the bridge system A3 (most regular configuration) is analysed; in Figure 7a, the envelopes of maximum, minimum and average displacements and drifts (negative and positive) along the longitudinal axis are reported. Differences in terms of average displacement or drift (positive and negative) in the order of 12 to 15% can be observed when comparing the response of the two different solutions (hybrid higher than monolithic). In the hybrid connection, the maximum displacement (mean value) is about 17 cm with a constant drift demand (1.7–1.8%) along the longitudinal axis due to its regularity in terms of distribution of bridge pier heights. Furthermore, in the system with hybrid connections, the maximum and minimum (positive and negative) displacement or drift envelopes look similar, while in the bridge system with monolithic connections a non-symmetrical behaviour is outlined. This fact is principally due to the higher self-centring capacity provided by the hybrid connections. Detailed results on the response under the EQ5 record (Cape Mendocino, 1992) are shown in Figure 7b in terms of moment-rotation at pier/foundation section of piers 1, 2. Although both piers have similar moment-rotation capacity, the hybrid system shows a more symmetrical hysteresis behaviour than the monolithic system.

When considering the response of the bridge system B3, the effects of the non-regular distribution of the pier heights are evident. It can be noted that, when referring to maximum displacement demand, no substantial differences can be observed, when compared to the response of the bridge system A3. Inevitably, due to the distribution of pier heights, a significant difference in terms of drift demand in the piers results, with the inelastic demand concentrated in the central shortest pier (average drift values of around 2.1%, Fig. 8a). More importantly, due to the irregularity of the bridge system B3, as illustrated in Figure 8b, the inelastic demand is concentrated in the central pier, leading to a more emphasized asymmetric behaviour of the monolithic solution when compared the self-centring hybrid connection.

For the bridge system C3, a non-symmetrical distribution of pier heights has been considered. Differences in terms of medium displacements/drifts (positive and negative) in the order of 15% between the two connections considering maximum and minimum values are observed, as illustrated in Figure 9a. Referring to the moment-rotation behaviour at the pier-to-foundation sections, here not presented for brevity, the inelastic rotation demand is concentrated in the piers n° 2 and n° 3 while the pier n° 1 remains in the elastic domain.

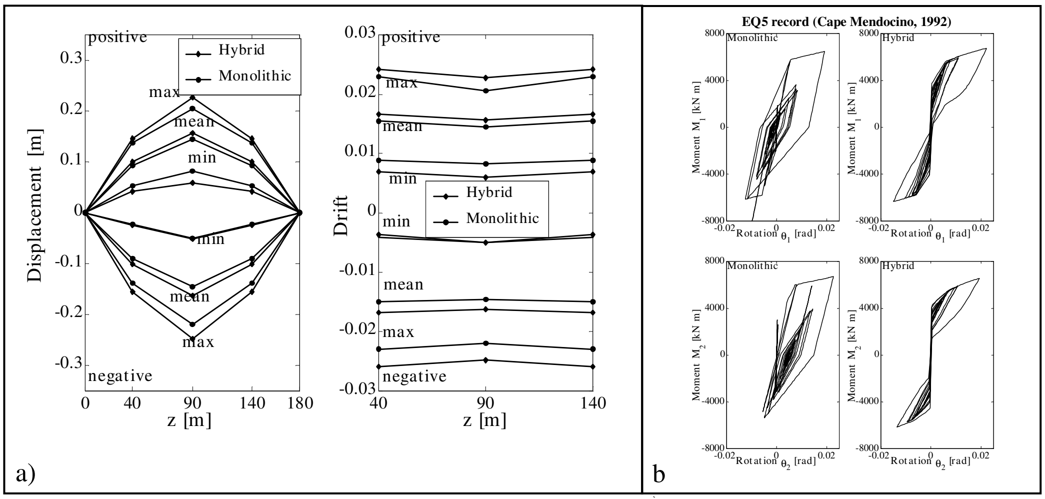

Figure 7. a) Bridge A3: displacement and drifts (maximum, minimum, mean values of 10 record) along bridge longitudinal z-axis; b) moment-rotation (pier/foundation section) of piers 1, 2.

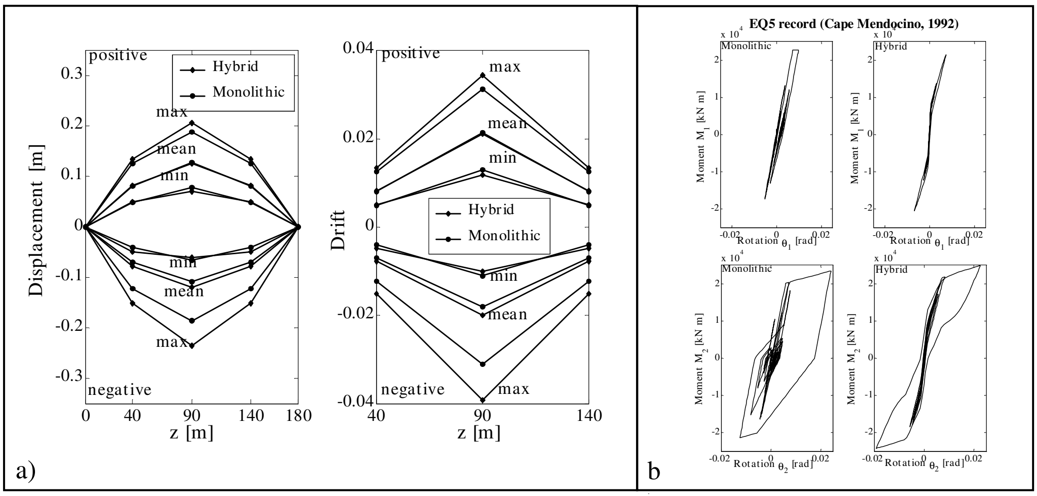

Figure 8. a) Bridge B3: displacement and drifts (maximum, minimum, mean values of 10 record) along bridge longitudinal z-axis; b) moment-rotation (pier/foundation section) of piers 1, 2.

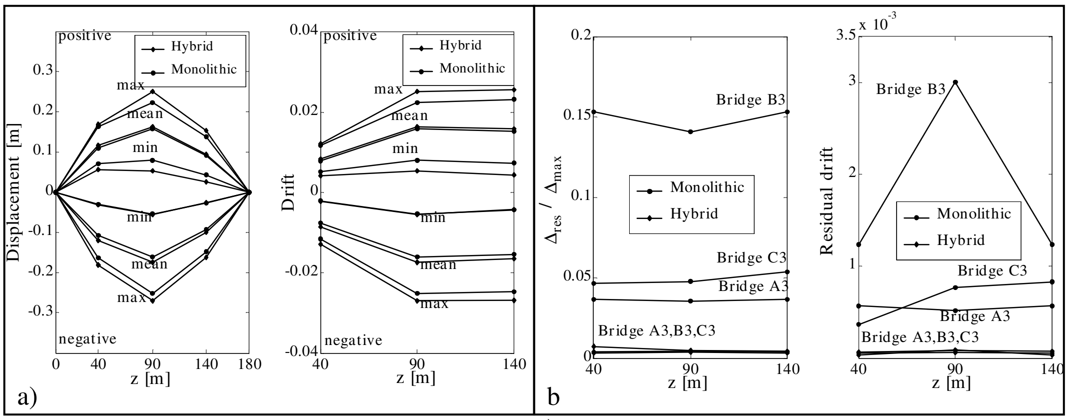

Figure 9. a) Bridge C3: displacement and drifts (maximum, minimum, mean values of 10 records) along bridge longitudinal z-axis; b) residual adimensional displacements and drifts (mean values of 10 records) along bridge longitudinal z-axis.

Finally, a comparison between the response of the bridge systems A3, B3, C3 is given in Fig. 9b in terms of mean values of residual drift as well as of the ratio between residual displacements/drifts Δ_{res} and maximum displacement Δ_{max} for each single pier, i.e. along the longitudinal axis of the bridge. In the case of monolithic connections, for irregular distribution of pier heights (bridge B3), the residual drifts are more marked (0.3%, pier n° 2) respect to bridges A3 and C3 (0.06-0.08%, pier n° 2). This fact is principally due to the concentration of inelastic demand in one single member (central pier) without self-centring properties. For monolithic connections, mean values of $\Delta_{res}/\Delta_{max}$ of about 15% are reached for irregular distribution of bridge heights (bridge B3), while negligible residual drifts are guaranteed by hybrid connections independently from the distribution of pier heights.

5 CONCLUSIONS

The extension and application of hybrid or "controlled rocking" concept to bridge piers has been proposed in this contribution as an efficient and promising alternative solution to traditional monolithic systems. The possibility of accommodating the inelastic demand at the critical section interface (pier-to-foundation and/or pier-to-deck) where a rocking motion takes place, clearly leads to a significant damage reduction (hence repairing costs) in the pier element. In addition, an inherent self-centring property, provided by the use of unbonded post-tensioning tendons/cable, guarantees negligible residual (permanent) deformations occurring after the seismic event.
Non-linear time-history analyses on different bridge configurations adopting monolithic or hybrid connections, have shown no substantial differences in terms of maximum displacements, with slightly higher values for the less-dissipative hybrid systems. More important, negligible residual displacement results in the response of bridges with "controlled rocking" solutions, while values in order of 20% of the maximum pier drift/displacement should be expected and accepted when monolithic connections are adopting. The irregularity of the system (different distribution of pier height) emphasizes the differences in terms of residual displacement more than those in terms of maximum displacements.

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