DAMAGE, ASSESSMENT AND STRENGTHENING OF THE VIADUCT 1 IN BOLU, TURKEY

M.J.N. PRIESTLEY and G.M. CALVI
ROSE School, Pavia, Italy.

1. INTRODUCTION

As part of the development of the Great Anatolian Highway, a series of viaducts are under construction in the mountainous region near Bolu, some 300 km from Istanbul. The first of these Viaducts, Bolu Viaduct #1 was essentially complete except for installation of expansion joints when the Duzce earthquake of 12 Nov. 1999 occurred adjacent to the bridge site [1]. Major damage occurred to the viaduct leaving the superstructure in an apparently precarious condition. After one year of internal discussions concerning the viability of retrofit and repair of the viaduct, the authors were contracted by the design/build contractors to develop conceptual schemes for reinstatement of the bridge, and upgrading to higher levels of seismic resistance. This paper summarizes these studies, and presents the final recommendations of the concept designers. In addition, an in depth discussion on the most appropriate choice for the isolation system and on the effects of considering the effects of vertical ground motion and of variation of the axial action on the isolating devices is presented, based on a number of non linear analyses.

2. EXISTING STRUCTURAL CONFIGURATION

Bolu Viaduct #1, part of the Great Anatolian Highway, consists of two parallel bridges, (Fig.1) each carrying a separate traffic direction, with the Ankara-bound bridge (right bridge) having 58 spans, and the Istanbul-bound bridge (left bridge) having 59 spans. Each span is approximately 39.4m long, and is constructed from 7 precast V-girders with an in – situ topping. Fig. 2 is a schematic of the typical cross section. Each span was supported on pot-bearings designed for 200mm maximum travel. A 1.5m long link-span extension of the deck slab connected the spans into 10-span segments with movement joints between the 10-span segments. However, because of the bearing support detail, the gap between the girder ends, and the flexibility of the link-slab, the spans remained effectively simply supported for live loads as well as dead loads (see Fig.3).

At internal supports, the bridge is supported by tall hollow reinforced concrete piers of approximately rectangular section modified by architectural detailing, generally in the range 40-50m high, though a number of shorter piers exist, particularly near the Istanbul abutment. Piers are numbered from P1 to P58 (or P59 for the left bridge), starting at the Istanbul end.

Piers are founded on massive reinforced concrete pile caps, in turn supported on twelve 1.8m diameter Cast-In-Drilled-Hole (CIDH) piles passing through surficial soils of variable strength and bearing on alluvium layers, generally at about 30m depth. Fig.4 illustrates pier and pile-cap details.

At the bridge ends, the V-girders were individually supported by pot-bearings on a seat-type abutment, again supported on CIDH piles, but of reduced number and size (11-1.65m diameter) compared with the internal supports.

Seismic resistance relied primarily on a seismic isolation system consisting of “crescent-moon” steel energy dissipating units (EDU’s) located at each support connecting the spans to a centrally mounted dissipator support block (EDU block). At movement joints and the central pier of each 10-span segment, the EDU’s incorporated sliders and lock-up pistons to allow relative thermal movements to occur freely, but to ensure full engagement of all EDU’s under seismic loading. Displacement capacity of the EDU’s was 480mm. Transverse displacements were restrained by shear blocks adjacent to beams 3 and 5 as a back-up in the event of extreme displacements, and longitudinal relative movements at expansion joints were constrained by cable restrainers.

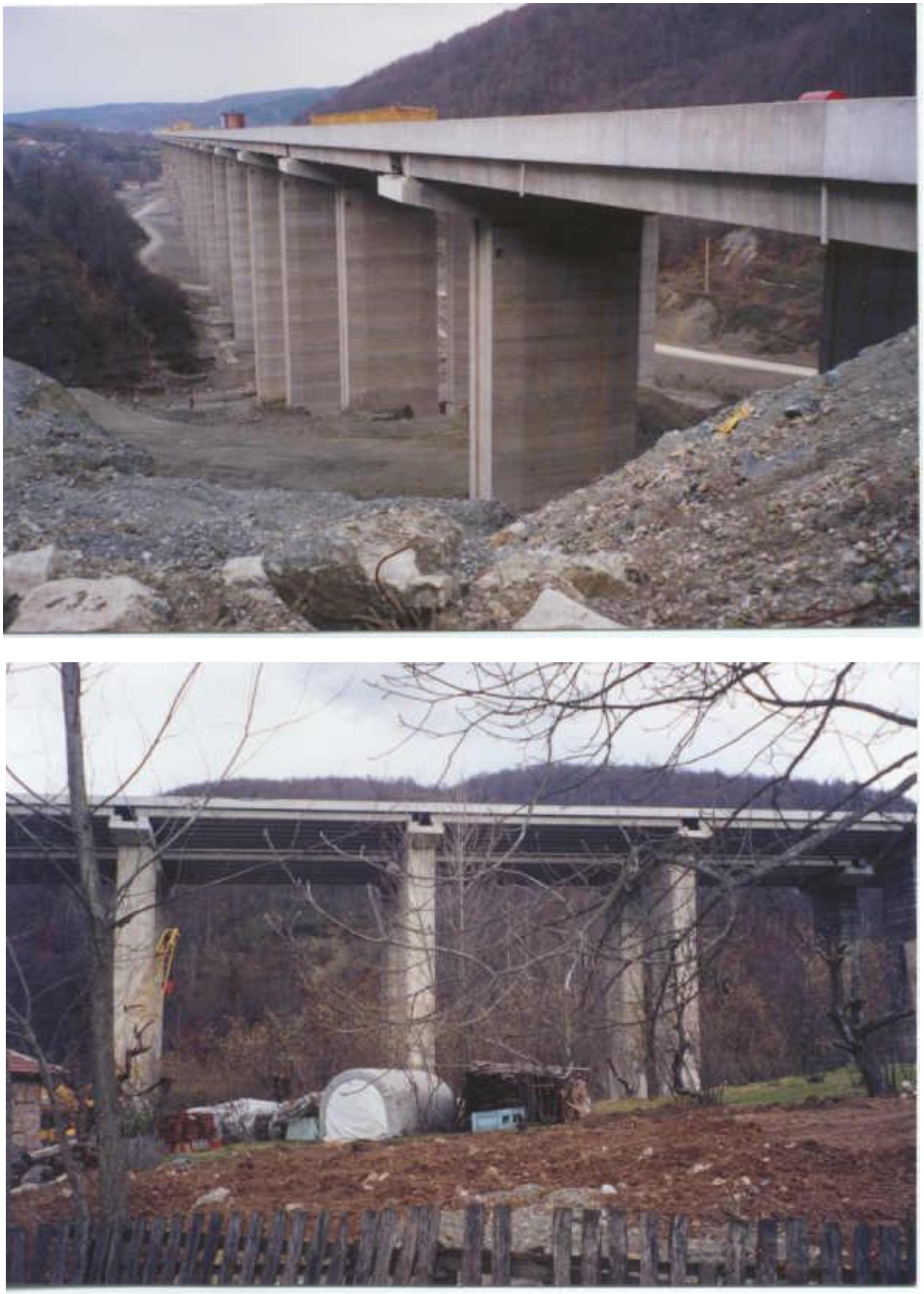


Fig. 1, – Two photos showing general views of the viaduct.

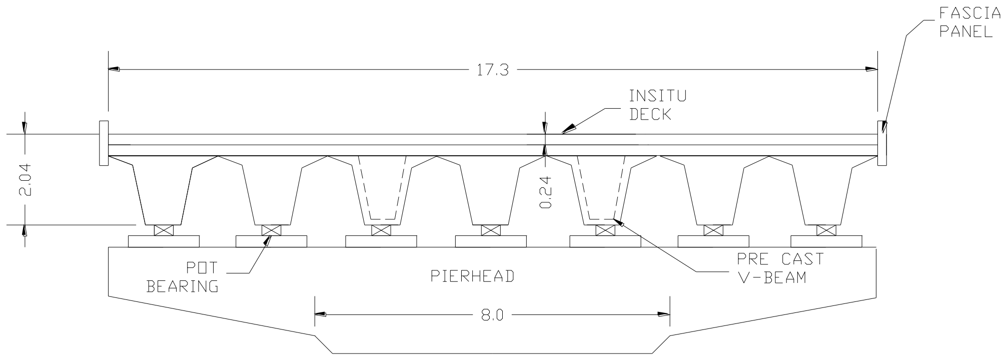


Fig. 2, – General cross section of the superstructure

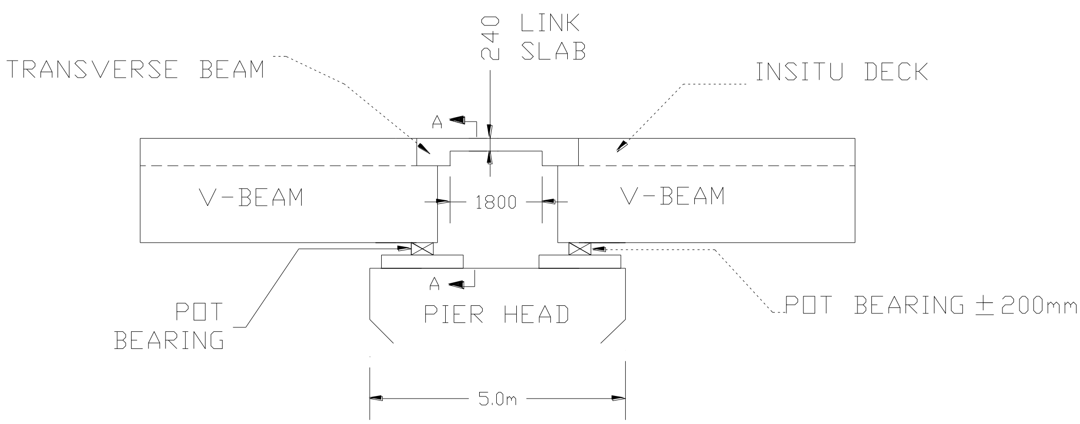


Fig. 3, – Side elevation of the link span/bearing support detail

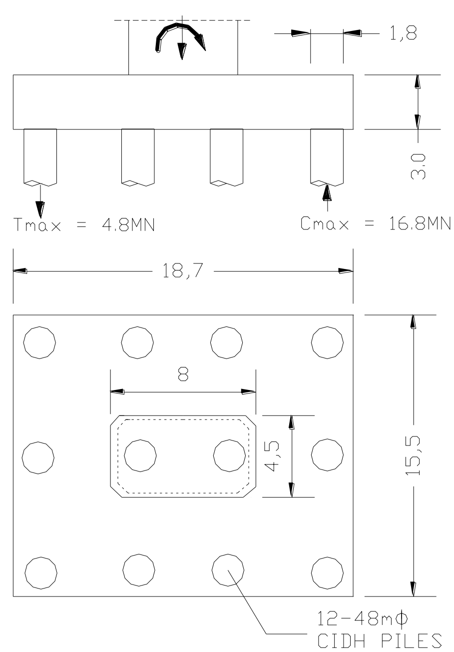


Fig. 4, – General dimensions of column section and pile cap (plan and elevation)

3. DAMAGE SUMMARY

On November 12, 1999 an earthquake of moment magnitude 7.2 occurred on the Duzce fault. This followed three months after the Kocaeli earthquake of August 17th, 1999, which caused extensive damage and loss of life. As well as causing damage to buildings, with the loss of about 1000 lives, the Duzce earthquake caused severe damage to tunnels and bridges under construction on the Great Anatolian Highway [1]. Peak ground accelerations in the vicinity of 0.8g, based on accelerograms recorded nearby (records were available in Bolu, 8 km from the viaduct site, and in Duzce, 7 km from the site) were estimated at the viaduct site. More important to the bridge performance, right – lateral fault slip of approximately 1.6m occurred on a fault scarp traversing the bridge alignment, at an acute angle (approximately 15 degrees to the bridge longitudinal axis), resulting in shortening of the bridge length by about 1.5m, concentrated over two spans of the bridge.

Displacements resulting from the fault slip and the vibratory response exceeded the capacity of the seismic isolation system. As a consequence, the EDU's were destroyed, (Fig.5), and the pot-bearings at most beam ends were ejected. Impact between the ends of the central V-beam (beam 4) and the EDU support block occurred at most spans, destroying many of the support blocks and damaging many of the beam 4 ends. In most cases the damage to beam ends was superficial, though in one case the damage included crushing of concrete and fracture of reinforcement for a distance of up to 4m from the beam support.

Impact between the transverse shear restraint blocks and beams 3 and 5 caused extensive damage to the shear blocks, and some damage to beam ends, though this was superficial. Further damage to beam ends, applicable to all beams across the section, occurred as a consequence of unseating from the pot-bearings, with impact between the beam end and the bearing support block, or the pier cap. This damage is minor in all cases.

As a consequence of the fault movement and the failure of the EDU's, residual displacement of the beam ends is considerable, being as high as 1100 mm longitudinally, and 500 mm transversely. In a number of cases this displacement is such that the beam ends are unsupported, hanging over the edge of the pier cap (Fig.6). In such cases, and where the beams ends are unsupported (having moved beyond the edge of the bearing blocks) but are still within the plan area of the pier cap, support for the beam end is only provided by flexure of the link span joining adjacent spans. The factor of safety against failure in these cases, using conventional flexural strength theory, is less than 1.0, since the shear corresponding to flexural strength of the link slab is only 90% of that needed to support the reaction of the span dead-load, using expected (rather than nominal) material properties, and ignoring strength reduction factors. It will also be noted that during unseating of the beam ends, the shear in the link spans will have been increased by dynamic impacts, to perhaps twice the static value. It is apparent that catastrophic failure has been averted by large vertical relative displacements, up to 300mm, which have occurred across these 1800mm long link spans, and the strength is apparently provided by a combination of flexure and tensile tie action in the link slab reinforcement, aided by the side fascia panels, which are deeper than the link span. This action is described in Fig.7.

Shortening of the bridge length as a consequence of the fault slip has largely been accommodated by reduction in the distances across the movement joints. Note that the movement joints had not yet been installed when the earthquake struck.

At the abutments, damage is similar to that at internal supports, being largely confined to EDU support blocks, transverse shear restraint blocks and beam ends. However, additional damage was caused to abutment back walls by impact as the bridge was driven towards Istanbul. This damage, consisting of wide horizontal and inclined cracks, only occurred at the Istanbul abutments (S1L and S1R).

The hollow reinforced concrete pier stems were largely undamaged, though a number of piers have small but significant tilts or rotations. In particular this affects pier 26R where tilt has resulted in a transverse displacement of the pier cap of about 600mm, and piers 45R and 47L, where the fault crossed the bridge. These piers have twisted about the vertical axis by approximately 4 degrees. Evidence of the pier twisting and superstructure lateral displacement is visible in Fig. 8.

Fig. 5, – Failure of an EDU, damage to EDU support block and Central beam end, due to impact

Fig. 6, – Unseating of Beam end at Pier cap

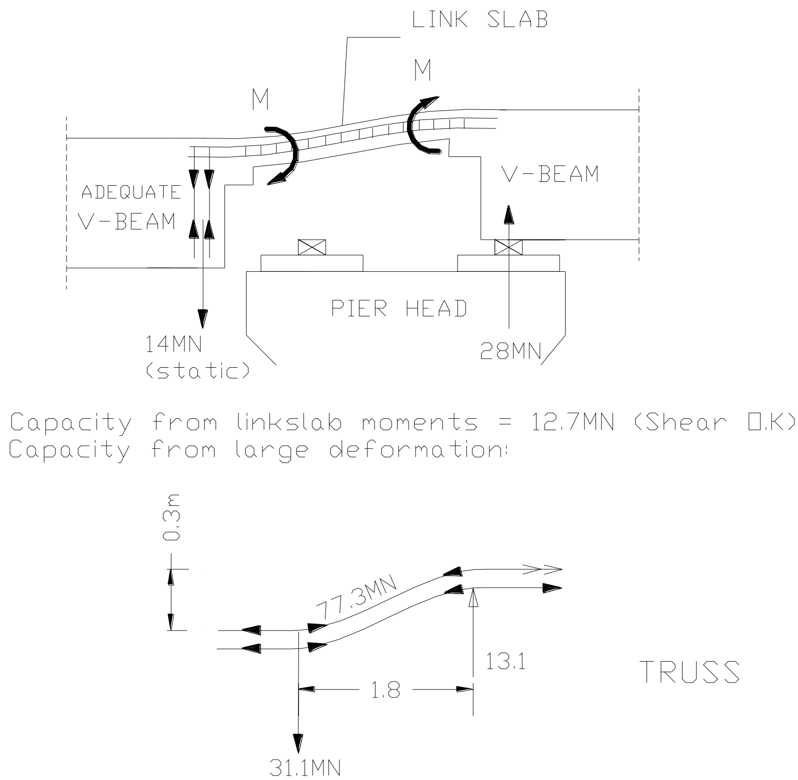


Fig.7, – Support of Span end by rotation and tie action of reinforcement in link span.

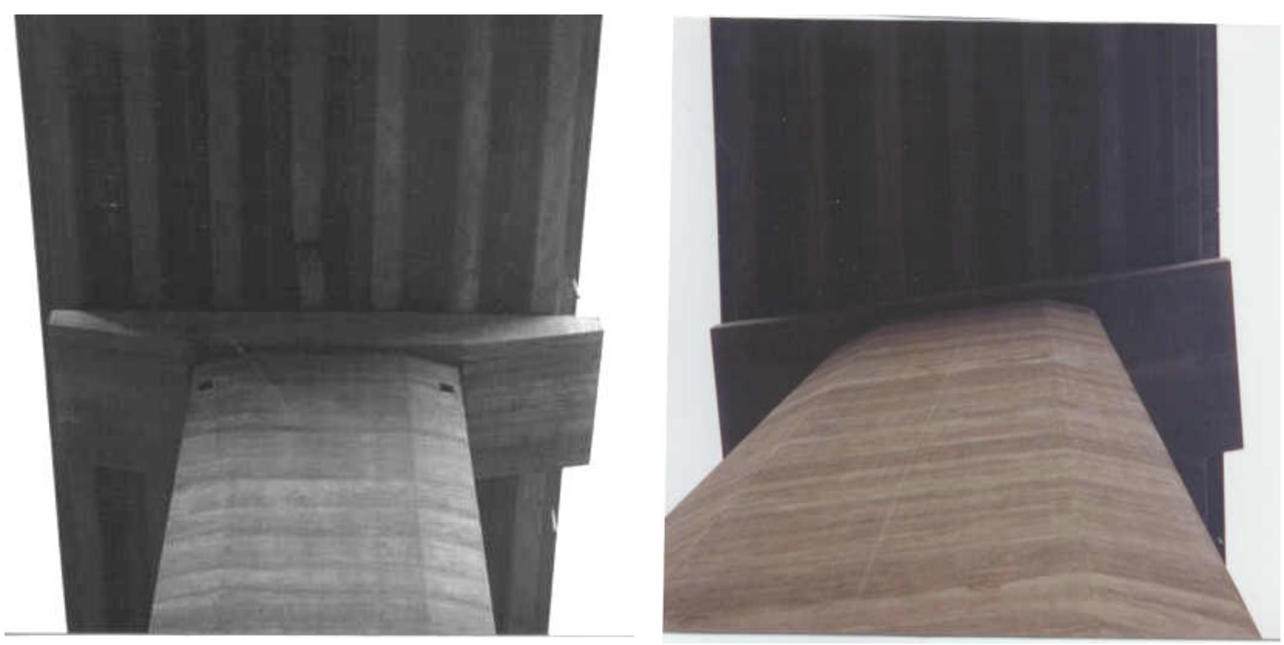


Fig. 8, – Evidence of pier twisting and lateral displacement of the superstructure in the vicinity of the fault line.

Fig.9, – Extreme example of damage to piles of a foundation on the fault line.

With six exceptions, damage to the foundations is minor. Damage to the six exceptions include significant pile cap cracks, and plastic hinging to the piles, as a consequence of gross ground displacements in the vicinity of the fault movement. Where the foundations were rotated by proximity to the fault, damage to the piles was often severe, as illustrated in Fig.9. In one case some distance removed from the fault (Pier 26R) the damage appears to be related to loss of lateral support in especially weak superficial soils during seismic response.

4. REPAIR/RETROFIT CONTROVERSIAL ISSUES

The condition of the Bolu Viaduct following the Duzce earthquake posed a number of controversial issues. On the one hand, the bridge was evidently in a state of near collapse, but on the other, damage was concentrated at span ends, and large portions of the bridge were essentially undamaged. It appeared that provided the bridge could be repositioned safely on its supports, repair would be much less expensive than removal and replacement. The high quality of construction also indicated a desirability of retaining the structure. Issues that needed to be addressed included:

- Could the bridge be safely and economically repositioned?
- Should the bridge be repaired to the initial design level of seismic safety, or to some higher level?
- Should the structural system be modified to reduce seismic risk?
- Could future fault dislocations be expected during the Viaduct's design life of 100 years, and if so, how could these be accommodated in the design?
- Is seismic isolation the correct seismic design strategy for the Viaduct?

The authors were employed to develop answers to these questions, and if possible to develop conceptual designs for repair and retrofit, in cooperation with the original designers, the project engineers, the construction company and the client – owner.

5. RECENTERING THE BRIDGE

Initial studies showed that it would be comparatively straightforward to reposition the bridge, though EDU support blocks, transverse shear restraint blocks and broken EDU's will need to be removed before repositioning occurs. Repositioning will be undertaken for an entire 10-span unit between movement joints at a time. The first operation involves lifting the unsupported or unseated span ends and placing them on temporary supports and low-friction sliders. The second operation involves moving the 10-span segment under consideration back to its original longitudinal position with respect to the piers. Note that exact repositioning is not possible because of small permanent deformations at pier caps relative to ground, and repositioning should thus aim to minimize errors for a 10-span segment. Calculations for sliding are based on a 5% coefficient of friction, and the proposed procedure uses two hydraulic jacks at each of three piers of the 10-span segment, thus using the piers as reaction points. Moments induced in the piers by the jacking process are a maximum at the reaction piers, but are small compared with seismic or thermal moments.

After correctly repositioning the segment longitudinally, it can be moved back to its final position transversely. Again, two jacks at each of three piers per 10-span segment can be used for this procedure, utilizing the stiff deck as a diaphragm between the jacking points. Figure 10 shows a final detailed sketch of the jacking system used to re-center the bridge.

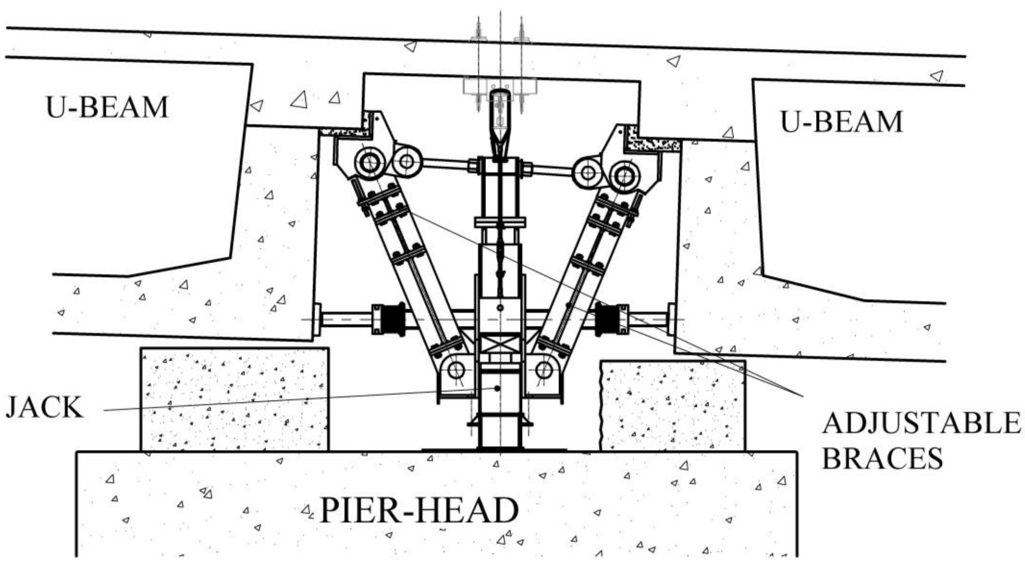


Fig.10, – Longitudinal section at the pier head, showing the jacking system to re-center the bridge

6. REASSESSED SEISMIC HAZARD

Initial seismic design was to a standard AASHTO [2] spectrum for a peak ground acceleration of 0.54g. The Duzce earthquake clearly indicated the potential for higher seismic intensity at the site. However, since the return period for earthquakes of that magnitude at that location of the Anatolian fault was estimated at several hundred years, it could be argued that with a 100 year design life, a smaller design earthquake could be contemplated. However, intensive site and theoretical seismological investigations indicated that the redesign input ground motion should be characterized by the following properties, characteristic of a 2000yr return period ground motion [3]:

- Design peak ground acceleration (PGA) 0.81g
- Design peak spectral acceleration (5% damping) 1.8 – 2.0g
- Design peak spectral displacement (5% damping) 600mm
- Consideration of near field directivity effects.

The latter point meant that fling effects (velocity pulses) and fault slip should be considered. A ground permanent deformation of up to 500mm was estimated to be possible during the design life of the bridge. Based on these assumptions, a set of fifteen horizontal and two vertical accelerograms were selected and used for all non linear analyses, also trying to satisfy the following additional conditions, which characterised the Duzce fault and the location of the viaduct:

- The magnitude of the earthquake should be of the order of 7-7.2 , consistent with the characteristic earthquake on the Duzce fault
- The earthquake fault rupture should be strike slip;
- The recording site should be located with respect to the epicentre in such a way that the angle between the fault and the line connecting epicentre and location is clockwise and small.

All the records were from near field locations and had been produced by source mechanisms reasonably similar to the strike-slips events typical of the Bolu area.

The set of accelerograms was derived from nine recordings of natural events, and two components were artificially generated. In table 1 the basic characteristics of all accelerograms are given; it my be noted that five pairs of orthogonal components and five single component records are included, and that the two vertical ground motions correspond to single component records. For this reason, the following sequence of sixteen non-linear analyses were performed:

a. The five couples of orthogonal components were applied simultaneously, approximately respecting the fault orientation (five analyses).
b. The single components records were applied simultaneously in two orthogonal directions (five analyses).
c. The two cases where the vertical motion was not available will be also run with the simultaneous application of the three components (two analyses).

Finally, the two cases for which the largest demands had been obtained, were run considering a non synchronous input motion, assumed to be produced by a time delay only, without consideration of loss of coherency and local soil effects. Four different values of time delay were considered, assuming a velocity of propagation equal to 400, 600, 800 and 1000 m/s (eight analyses).

Table 1 – characteristics of the ground motion used for the non linear analyses

#	Earthquake	Magnitude	Components			Site conditions	Recording distance (km)	PGA (cm/s^2)	Filter
			L	T	V				
1	Bolu	7.4	X	X		Deep alluvium	8.0	790 - 727	High pass 0.15 Hz
2	Motoyama	6.9	X	X		Stiff	3.3	380 – 690	Band pass 0.1 – 10 Hz
3	Bonds Corner	6.5	X	X		Alluvium	2.4	576 – 770	Band pass 0.03/0.17 – 23/25 Hz
4	Corralitos	7	X	X		Rocklike	3.4	618 – 479	Baseline
5	Artificial	7.2	X	X		-	-	810	-
6	Lucerne	7.1		X	X	Soil and alluvium	1.1	854	Band pass 0.1 – 10 Hz
7	Sakaria	7.3		X		Stiff soil	3.0	417	Band pass 0.1 – 10 Hz
8	Kobe JMA	6.9		X	X	Medium – stiff	3.4	817	High pass 0.15 Hz
9	Fukiai X	6.9		X		Alluvium	1.6	688	High pass 0.10 Hz
10	Shin Kobe	6.9		X		Rocklike		509	High pass 0.10 Hz

The 5% acceleration and displacement response spectra for the fourteen records are compared in Fig.11. It will be noted that the records encompass a wide range of possible response. In some cases the spectral acceleration at periods less than 0.5s and spectral displacement at periods in the 2 – 3.5s period range significantly exceed the design specifications.

Clearly these seismic design criteria, which represent intensity levels more than 100% larger than envisaged in the initial design indicated the need for careful structural analysis with a potential for redesign.

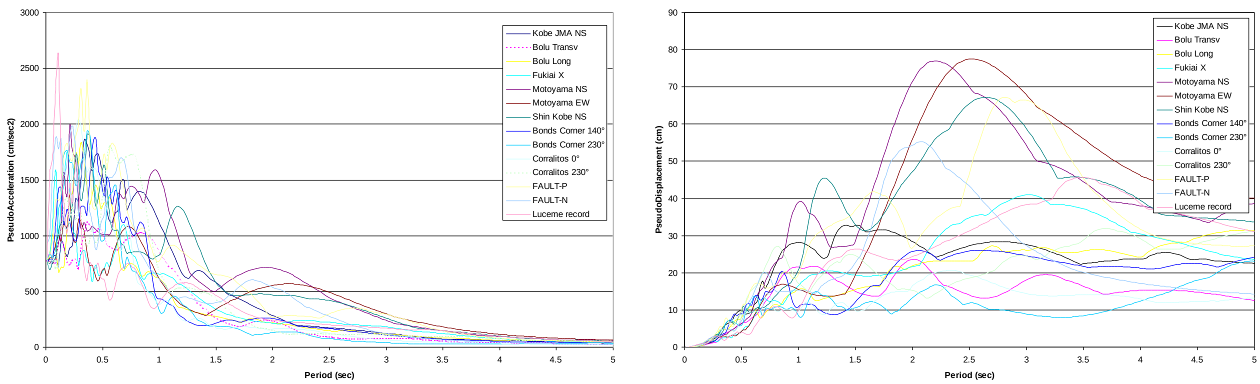


Fig.11, – Comparison of acceleration and displacement response spectra of design accelerograms

7. REASSESSED STRUCTURAL SYSTEM

7.1 Bearing Design

Preliminary redesign focused on making the 10-span bridge segments fully continuous and using reduced numbers of large capacity (axial force and displacement) seismic isolation bearings. After consideration of alternative bearing designs, the client expressed a preference for friction pendulum isolation bearings.

It was immediately apparent that the existing detail for supporting the simple spans on the pier heads would not provide adequate displacement capacity for the enhanced design seismic intensity, and that continuity over supports would be necessary. It was also apparent that seismic isolation could result in problems. Normally it can be assumed that force levels can be kept to a desired maximum level by seismic isolation, at the penalty of increased displacements. In the case of the Bolu viaduct, however, the mass of the piers is typically larger than that of the superstructure, as a consequence of the tall piers, and short spans (20.5MN compared with 14MN for the tallest piers). With a seismic isolation system placed between the superstructure and the pier head, only the superstructure mass is isolated, and pier moments resulting from pier self-mass are unreduced. As the seismic intensity increases, the pier moments and shears must therefore also increase. It was clear that pier design forces would thus be much larger than in the original design.

A second concern with seismic isolation was the level of displacement that might develop between deck and pier head. In the normal seismic isolation approach, bearing displacements will be less than the 5% spectral displacement for the isolation period, as a consequence of structure flexibility and additional damping provided by the isolation system. The displacements would be much less than the peak 5% spectral displacements (600mm in this case). However, with the high pier mass, the response is essentially a two-mass system, and in the second mode the pier head and the superstructure move out of phase. In this case the bearing displacement can be significantly higher than the spectral displacement for the second mode period. With a second mode period initially estimated at about 1.2s, 5% spectral displacements as high as 400mm could be expected. Also, it was clear that the argument related to the first mode, expounded above, was a simplification, since the high mass of the pier would result in the effective height being some distance below the superstructure, and hence the bearing displacement could still exceed the spectral displacement in the first mode alone.

These qualitative considerations were confirmed by the results of dynamic response analyses, both considering a single pier response and the global bridge configuration.

A third concern related to the required bearing design displacements, when tectonic fault movements were considered. Relative displacement across the fault is expected to develop at a rate of about 5 mm per year, leading to the design relative fault movement of about 500 mm in the 100 year design life of the bridge. With the fault locked until fracture occurs, the question arises as to how will the fault movement be distributed with distance from the fault (which passes through the bridge). Clearly two points some kilometers away from the fault on either side will move relatively by the fault movement, but two points a few meters on either side of the fault will essential experience no relative movement until the fault ruptures. On the other hand, the two points close to the fault will experience the full fault dislocation during the fault rupture as an essentially instantaneous relative displacement, whereas the two points kilometers from the fault will see no additional relative displacement during the fault rupture.

It is of relevance to the bearing design in the Bolu viaduct to know how the tectonic displacement is distributed with distance from the fault. If the relative displacement develops rapidly with distance from the fault (say within a few spans of the bridge from the fault), then most of the bearings will need to be designed to accommodate the additional displacements due to tectonic movement. However, if the tectonic displacements develop only slowly over a number of kilometers from the fault, then until the fault fractures, the piers of the bridge will maintain their current relative locations as tectonic movements develop. When the fault ruptures, relative displacements will develop only in the immediate vicinity of the fault, affecting bearings in the immediate vicinity. The two alternative scenarios would result in the difference in designing all the bearings for displacement capacities of about 900mm (600mm vibrational and 250mm fault slip) or most of the bearings for 600mm (vibrational only) with the bearings immediately adjacent to the fault having the increased 900mm capacity. There was a significant financial implication dependent on the resolution of this question.

Theoretical considerations supported slow development of tectonic relative displacement with difference from the fault. These were supported by measurements of relative positions of the pier bases taken before and after the 1999 earthquake. These measurements indicated that the abutments moved closer together by an amount at least equal to the fault dislocation of 1.5 m. This could not have happened if the ground at the abutments had already been displaced by the full (or a significant part of) the relative tectonic displacement. As a consequence only a small number of bearings would need to be designed to accommodate the sum of vibrational and dislocation displacements.

It could also be argued that it is unlikely that full vibrational and dislocation displacements will be additive, as it implies that (a) the dislocation occurs before the vibrational peak (which may be possible, but is uncertain), and (b) that the vibrational response, which is dependent on the development of resonance, is unaffected by the fault dislocation. It would seem probable that the process of fault dislocation would act to damp out vibrational response. As this controversial issue could not be resolved with certainty, the bearings in the vicinity of the fault are to be designed for the full combination of vibration and dislocation.

As a consequence of these initial concerns, it appeared that seismic isolation bearings could be subjected to very large displacements under the revised seismic hazard, particularly when considerations of potential fault slip or creep were included.

7.2 Pier and foundation capacity

At an early stage, the typical pier moment and shear capacities were assessed, since it was obvious that the forces developed under the revised seismic hazard would exceed the original design forces, as discussed above. A further reason to investigate pier capacity was the consideration of an alternative approach to seismic isolation, where the superstructure would be pinned to the pier heads. Although this could be expected to cause problems for the shorter piers, it was expected that the displacement capacity of the taller piers would exceed the maximum that could possibly develop in the design level intensity.

Moment-curvature analyses were carried out to determine capacities corresponding to different limit states, using material strength data recorded during construction, and without using strength reduction factors. Strain-hardening of reinforcing steel and the effects of concrete confinement were included in the analyses. The results of these analyses indicated nominal moment capacities in the longitudinal and transverse directions of approximately 600MNm and 1000MNm respectively, with limit state curvatures given in Table 2.

Table 2 – Limit state curvatures obtained from moment – curvature analyses of the pier section

Limit State	Longitudinal Curvature (mm^{-1})	Transverse Curvature (mm^{-1})
Nominal Yield	1.17	0.67
Serviceability	4.06	2.42
Damage control	15.00	6.50

Serviceability limit strains were taken as 0.004 and 0.015 for concrete and reinforcement respectively, with damage control limit strains of 0.01 and 0.05, though calculations indicated the ultimate compression strain of the confined concrete to be about 0.017 [4].

Shear strength was assessed using the method described in [5], which tends to be considerably less conservative than codified approaches. This resulted in a typical shear capacity at the base of the piers of 45MN for limited ductile response, corresponding to more than 1.5g on the total supported mass.

The moment and shear capacities of the piers were found to be substantially higher than assumed in the initial design, and were expected to be adequate for either an isolated or pinned pier/superstructure connection detail.

Limit state displacements were calculated from the curvature limit states for different pier heights, obtaining that for pier heights greater than 40m (about 90% of the piers), longitudinal and transverse displacement capacity would exceed 1.0m and 0.6m respectively at the serviceability limit state, for which no repair would be needed. Since these displacements exceeded the expected seismic displacements for the pinned connection case, it was apparent that this approach could be viable for the majority of the piers.

Assessment of the foundations and piles indicated that the structural capacity would be adequate to support the pier moment capacities, but that pile uplift would occur at about 70% of pier capacity. As a consequence, under the design event, a form of seismic isolation would occur as a consequence of soil limitation on foundation capacity. Although the structure would be essentially undamaged if this occurred, there was some possibility of undesirable residual displacements due to tilting.

7.3 Superstructure retrofit

As indicated above, an essential part of the retrofit/redesign was the decision to make the 10-span segments of superstructure between movement joints fully continuous. This was not a particularly controversial issue, and will not be described in detail here. The process of creating continuity will involve casting a new prestressed diaphragm beam at each internal support, of sufficient width to capture the end 600mm of the beams of the two adjacent spans. Longitudinal continuity between the beams and the new diaphragm beams is achieved partly by shear friction, based on the effective prestressed force, and partly by dowels drilled into the beam end and side faces. At internal supports, the diaphragm width is 3.6m, resulting in a significant additional mass to the superstructure (about 10%). Two isolation bearings would be inserted between the diaphragm and the pier head. Fig.12 shows the side view of the concept. Placing the diaphragm beam results in very little difference to the way dead load is supported. Live loads are supported by fully continuous action, well within the shear friction capacity of the beam/diaphragm connection. The critical design case for the connection was found to be differential thermal effects resulting from diurnal temperature fluctuations in midsummer.

At movement joints, separate prestressed diaphragm beams of reduced width are provided at each segment end.

7.4 Foundation/footing retrofit

For footings that were undamaged, or subjected only to minor cracking in the Duzce earthquake, no retrofit or repair will be undertaken, despite the overturning capacity being less than the flexural strength, which violates capacity design principles. As discussed above, foundation rocking, utilizing pile pull-out, can be considered a form of seismic isolation, with significant damping potential and with little possibility of undesirable consequences. With the seismic isolation bearing solution, foundation forces would not reach the level where rocking would initiate.

For the few footings where significant damage occurred, repair and retrofit is needed. The design philosophy has been to accept the capacity of the existing damaged piles to support dead load (which it is evident that they currently possess), and to provide a footing overlay with side extensions and additional piles to act in conjunction with the existing footing (but not the existing piles) to support the seismic forces; figure 13 shows details. In the past, the approach taken in designing footing extensions is to expose the existing footing bottom reinforcement mat by chipping the cover concrete back for about 400mm, then welding extension bars to the existing footing reinforcement to provide the necessary continuity. This is expensive and difficult where the base of the footing is below the water table, as is the case in Bolu. An alternative solution, where the axial compression forces in the new piles, which govern the design of the footing extension, are carried by strut–and–tie action within the new side extensions has been developed for the Bolu viaduct. The process carries the forces round the corner of the pile cap extension using headed reinforcement hanger bars in the corner regions. Continuity between the existing and new concrete is provided by dowel bars that act only in shear.

7.5 Additional repair and retrofit measures

Additional repair measures are required at abutments, and at damaged beam ends. As the issues involved are not particularly controversial, they will not be described herein. Special seismic lock-up devices were designed to be placed across internal movement joints between segments to reduce relative seismic displacements, and improve seismic performance. These were not placed in the movement joints adjacent to the fault trace. Massive shear keys were also designed into the diaphragms at the segment ends to restrict relative transverse displacement. Again these were not placed at the movement joints adjacent to the fault. Simple steel sliding plate movement joints were designed for the bridge/abutment movement joints, and for the movement joints on either side of the fault trace.

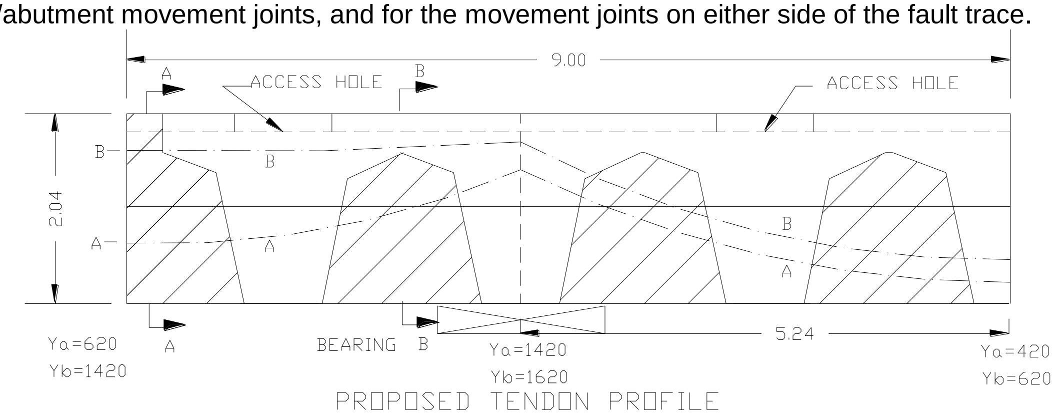


Fig. 12, – Prestressed diaphragm beam over internal supports for continuity

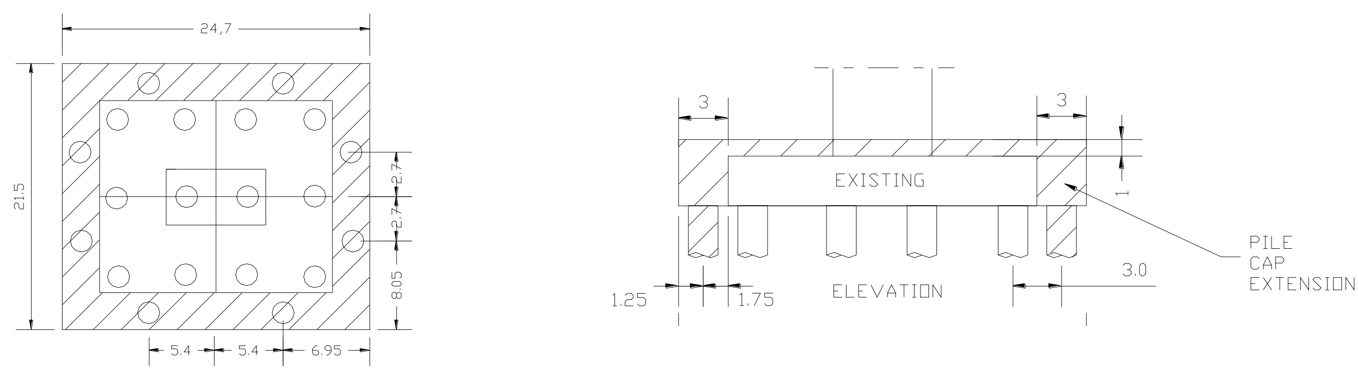


Fig. 13, – Footing enlargement for damaged footings

8. DYNAMIC ANALYSIS RESULTS

Verification of the repair/retrofit design concepts was carried out using inelastic time-history analyses, using an in-house modified version of the computer program FEAP [6]. Since the piers were expected to respond essentially elastically, they were modeled with elastic members of stiffness appropriate to the expected maximum response level, considering the influence of cracking.

Bearing response was modeled by bi-linear hysteretic rules. This is clearly not adequate to a proper representation of the response of bearings with equivalent yielding and post yielding stiffness linearly depending on the vertical force, such as the friction pendulum devices. Considering the couple of devices positioned on top of each pier, during the earthquake response the vertical force in each one of them will vary, both because of rotational equilibrium and of vertical acceleration. Although the relevance of these effects on the global response of an essentially straight bridge may not be significant, this problem has been the subject of proper in-depth studies, as discussed in the following section.

Three different structural configurations were analysed under the accelerogram sets discussed previously. The first (ISO) represented a fully isolated bridge with isolation bearings at all piers and at abutments. The bearing force-displacement response characteristics are defined as following:

Total design displacement (d_t)	$\geq$ 900 mm (type a)
	$\geq$ 700 mm (type b)
Expected maximum vertical load	11,100 kN (a, b)
	5,650 kN (a', b')
Yield force (assuming a bi–linear equivalent response, F_y)	$F_y \cong$ 400 kN (a, b)
	$F_y \cong$ 200 kN (a', b')
Force corresponding to d_t□ (F_{max})	$F_{max} \cong$ 1500 kN (a)
	$F_{max} \cong$ 1300 kN (b)
	$F_{max} \cong$ 750 kN (a')
	$F_{max} \cong$ 650 kN (b')
Equivalent damping ratio	$\geq$ 20 %

In the above list, a and b refer to internal bearing locations adjacent to and distant from the fault trace respectively, and a' and b' refer to corresponding locations for segment-end bearings, which carry half the internal bearing axial forces.

Existing movement joints at Piers 10 and 30 were eliminated. At piers 20, 40 and 50 the bridge segments on either side of the movement joints were each supported by isolation bearings on the piers, but were free to displace transversely and longitudinally relative to each other.

The second structural configuration had all piers and abutments, pinned to the superstructure. In this configuration (FIXED) the deck and pier head displacements were the same.

The third configuration (ISO 30-) represented a partially isolated structure, where isolation bearings were placed at both abutments, and at all piers with a height of 30m or less. This required nine piers to be isolated: seven in segment 1 [S1 to P10] and two in segment 6 [P50 to S2].

Key results for maximum pier bending moments, shears and displacements, taken as the average of 10 inelastic analyses using combined longitudinal and transverse excitation, showed the following main features (more details can be found in [7]).

Bending moments induced in the piers can be compared with the nominal longitudinal and transverse moment capacities of 600MNm and 1000MNm respectively, discussed earlier. Abutment longitudinal moment capacity was much lower, at about 40MNm and 300MNm longitudinally and transversely. In general, moments for the fully isolated case were well below moment capacities except at the abutments, where longitudinal moments slightly exceeded capacity, indicating a low ductility demand, well within the serviceability limit of abutment response.

Over the central part of the bridge, longitudinal moments for the FIXED case exceed the ISO case by about 50%, but transverse moments are more than twice as high. However, apart from the abutments, and piers adjacent to the abutment, moments are within the nominal capacities. Isolating only the shorter piers (ISO 30-) reduces the critical moments at the bridge ends, without significantly increasing the central pier moments. However, significant increases are experienced in the longitudinal moments for piers in the 30 – 40 m height range, and it is clear that for this option to be adopted, it would be advisable to isolate all piers less than 40m in height.

Similar conclusions apply to the shear forces, though the differences between the three cases analyzed are less pronounced, except in the end segments. In a few cases, pier shears in the isolated bridge are higher than in the fixed case.

Displacements are of particular interest. Bearing displacements for the ISO case, not including the possible contribution from fault rupture. show maxima values at about 600mm, being therefore satisfactory for the design displacement capacity of 700mm.

Displacements at the top of the pier head for the three design cases showed that for the fully isolated case, all pier top displacements are significantly less than the nominal yield displacement. For the fully fixed condition, some longitudinal displacements for the shorter pier exceed the yield displacement by a small margin, but all longitudinal and transverse displacements are less than the serviceability limit state displacement. For the partially isolated case, all displacements are less than the nominal yield displacement.

Additional analyses were carried out with coherent non-synchronous input, with wave propagation at 400, 600, 800 and 1000 m/s. Except for the slowest velocity of wave propagation, which was felt to be unrealistic, there was no significant increase in bearing displacements, or pier or foundation forces.

A fully isolated solution has been adopted for the final design.

9. EFFECTS OF AXIAL FORCE VARIATION ON THE EXPECTED RESPONSE

As pointed out in the previous section, an appropriate modeling of friction pendulum systems (FPS), requires consideration of the effects of the axial force on both equivalent yielding force (due to the variation of the friction force) and second stiffness (since the equivalent tangent stiffness is equal to the supported weight divided by the radius of curvature), as described in figures 14 to 17.

To investigate the problem, a special finite element has been developed and implemented, performing a large number of parametric studies, presented in [8].

The general conclusions of that study, show that the inclusion of axial force effects may not be significant for what concerns variation of the displacement demand, but may induce important increment of shear, bending and torsional moment demand on the piers.

The fundamental parameters that may amplify, or reduce, these effects are the ratio between deck and pier mass, the aspect ratio of the deck, the radius of curvature of the bridge, the intensity of the ground motion and the consideration of vertical input, as briefly discussed below with reference to the Bolu viaduct.

- Ratio between deck and pier mass: a significant variation of the shear force transmitted from the deck to the pier may result in strongly attenuated effect at the pier base when the ratio of the pier mass to the deck mass is high. This is clearly the case for the Bolu viaduct.
- Aspect ratio of the deck: as shown in figure 18, for the same level of horizontal force, the axial force variation possibly induced by the horizontal acceleration is higher for a deck section relatively larger and for devices relatively closer one to each other. The case of Bolu is again favourable, with a ratio between bearing distance and horizontal forces couple around 5.
- Radius of curvature of the viaduct: it is shown that a curved bridge may result in higher effects, due to the interaction of vertical and horizontal response. The Bolu viaduct is relatively straight.
- Intensity of the ground motion: relatively high horizontal peak ground accelerations may induce more significant effects, like in the present case, where a PGA in excess of 0.8 g is assumed.
- Consideration of vertical input: the inclusion of the vertical component of the input ground motion may result in being the crucial point to verify whether important effects have to be expected and considered, particularly in the case under consideration, which does not appear to be particularly sensitive to other factors, as discussed above.

It is thus clear that the set of accelerograms used for design is not appropriate to perform an analysis of the viaduct including a proper simulation of the axial force effects on the isolation system. For this reason, the viaduct has been submitted to the same set of ground motions used in [8], scaled to 0.81 g on the larger horizontal component, for the two cases of considering and neglecting the axial effects.

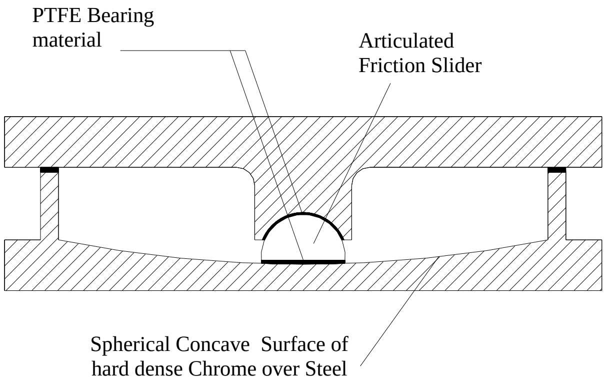


Fig. 14, – Cross section of a FPS device

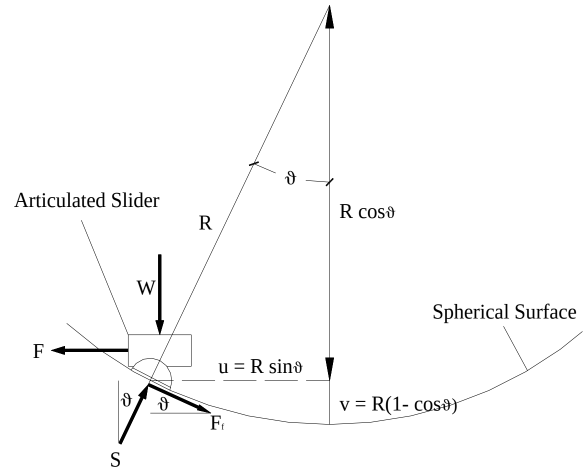


Fig. 15 – Equivalent static scheme of an FPS

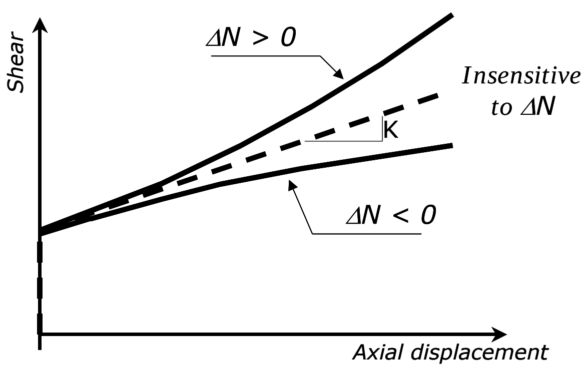


Fig. 16, – Variation of the second stiffness

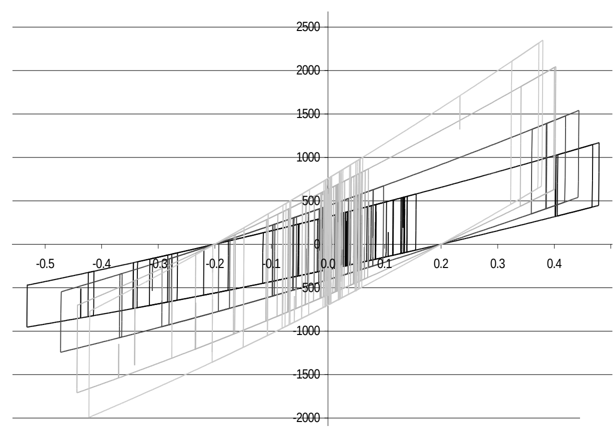


Fig. 17, – Variation of the hysteretic loops

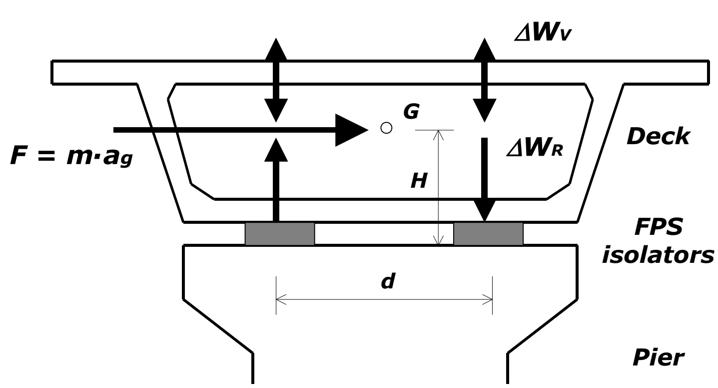


Fig. 18, – Axial force variation on the isolators at the top of a pier induced by a horizontal force, as a function of the aspect ratio of the deck

The results obtained generally confirm what was intuitively expected, as a consequence of the studies presented in [8].

First of all, the general trend to conserve the displacement demand was made evident, with no significant variation in the displacement of the isolation system.

As depicted in figure 19 for some selected piers of various height, the variation in the shear action at the top of the piers may be quite significant, with increments of the order of 20 – 30 % in the longitudinal direction and of 30 – 40 % in the transversal direction. However, this effect is severely reduced when the critical values at the base of the piers are considered, in which case a maximum average increment equal to 7 % is obtained.

This effect is due to the very high mass of the pier that implies a high shear demand on which the presence of an isolation system does not have any effect. Actually, as shown in figure 20, the ratio between the shear action at the top of the pier and the corresponding value at the pier base in never greater than 40 %, when the effects of axial force variation in neglected, and may rise to 50 % in case of use of a more accurate model. This aspect of the response of the viaduct was already noted in the studies to design the strengthening intervention, and used as a possible argument against the isolation of the tallest piers [7].

These considerations on the shear demand results in analogous comments on the bending moment demand, shown for the same selected cases in figure 21: it may be noted that maximum average increments of the order of 10 % are obtained.

While on one side these results are reassuring for what concerns the response of the Bolu Viaduct, on the other they are also confirming the general trend of potentially significant increments in the shear demand as a result of the effects of axial force variations in friction pendulum devices. This will require further studies to properly define the limit parameters for which these effects have to be included in the simulation of the response and may imply the necessity of applying capacity design principles in the design of the piers of new bridges, to avoid possible shear failures.

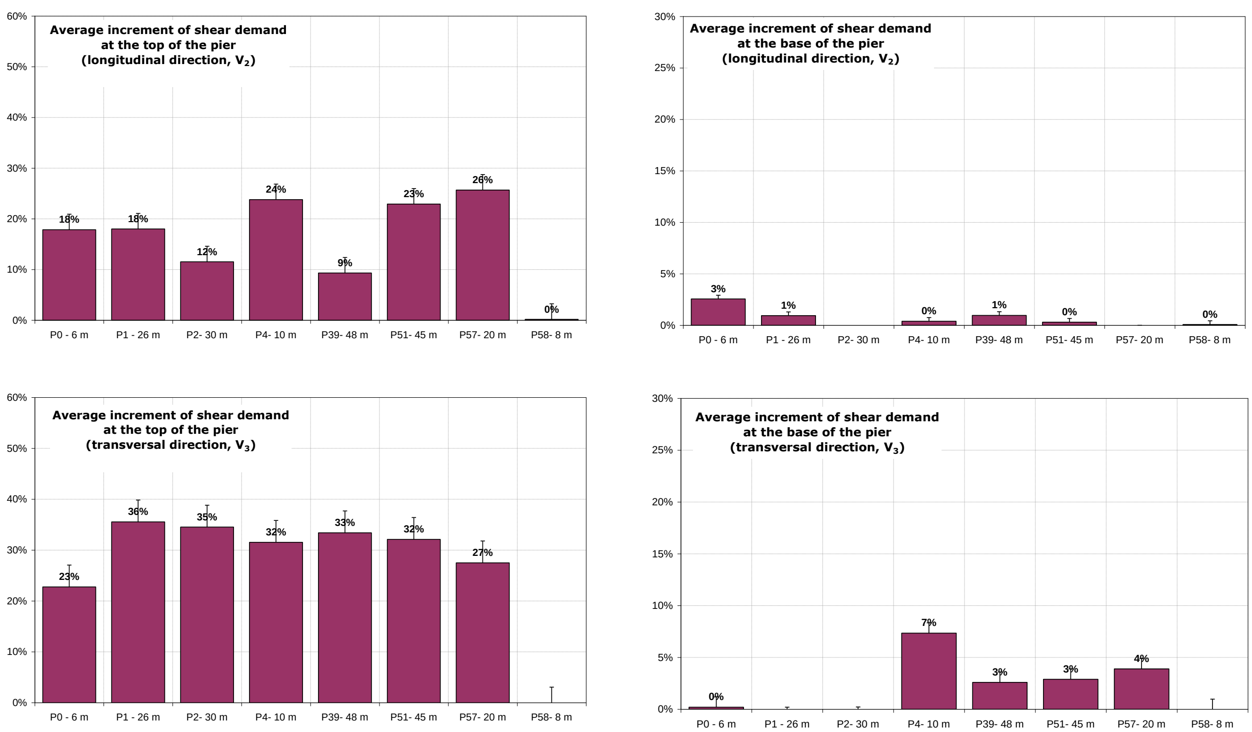
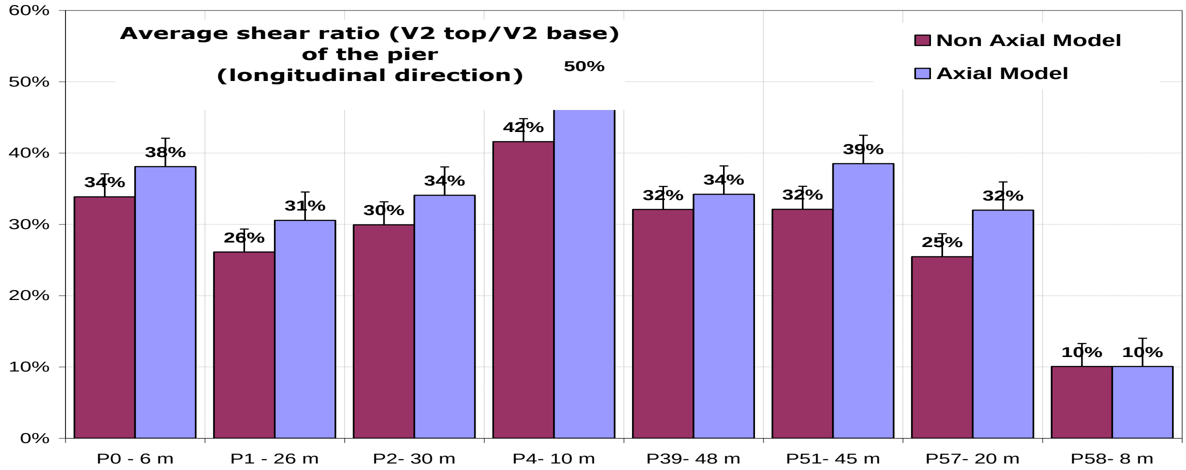


Fig. 19, – Average variation of the shear demand in the longitudinal (upper diagrams) and transversal (lower) directions, at top (left) and bottom (right) of some selected piers



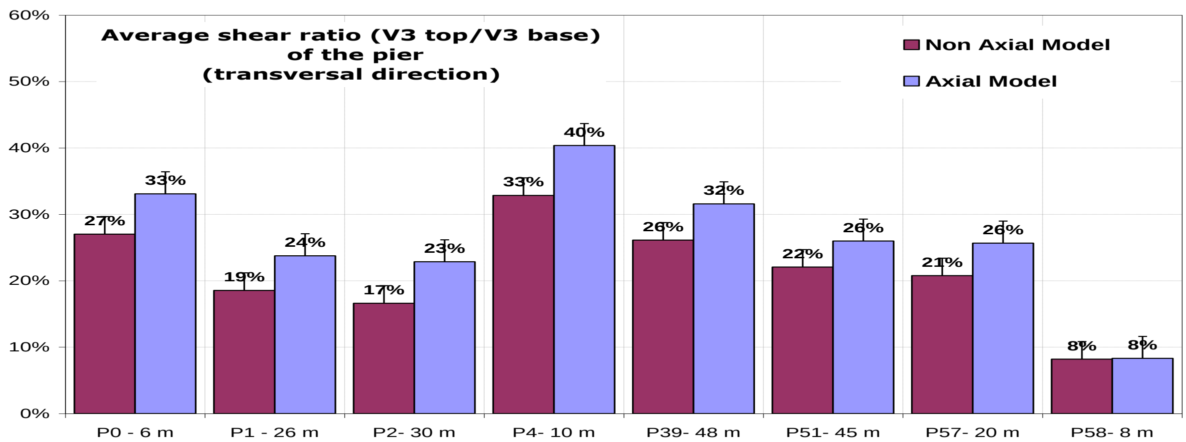


Fig. 20, – Average ratio between the shear demand at the top and bottom of the piers, in the transversal and longitudinal direction, neglecting and including the effects of axial force variation

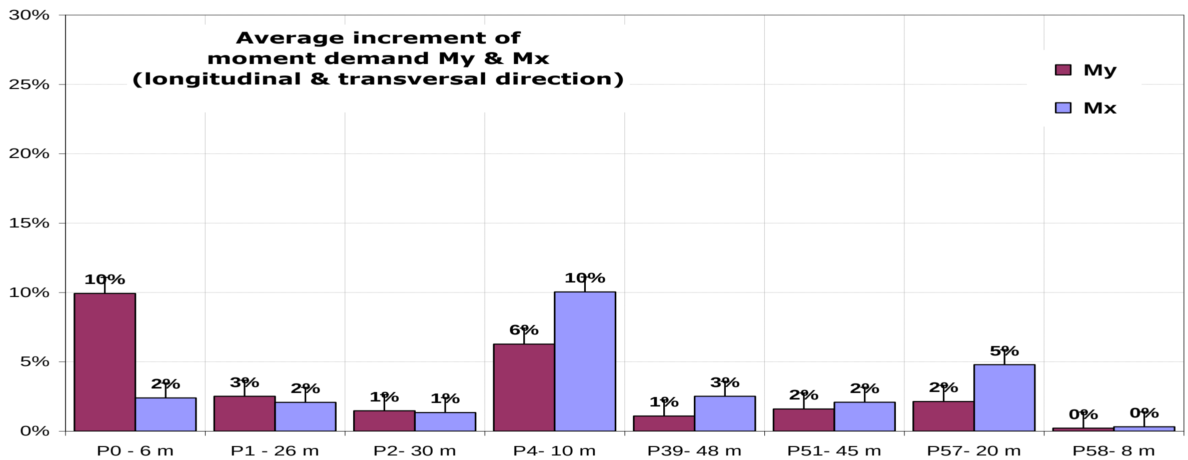


Fig. 21, – Average variation of the bending moment demand at the pier base, in the transversal and longitudinal direction

10. CONCLUSIONS

The review of the Bolu Viaduct for repair and seismic upgrading posed a number of controversial issues, for which firm conclusions were required since the study related to the “real world” rather than idealized academic conditions. The design brief required reinstating a damaged bridge to a level where it could respond within the serviceability limit state to earthquakes of intensity more than twice the original design earthquake, and larger than the Duzce earthquake which nearly caused collapse of the bridge.

Simple solutions for repositioning the superstructure were developed, and a change in structural configuration from simply supported spans to 10-span continuous segments was proposed.

Time-history analyses of different degrees of seismic isolation indicated that isolation was not essential for the regular central portion of the bridge, where displacements and pier forces would be within serviceability limit states, even for a fixed pier/superstructure configuration. However, isolation would be essential for the abutments and for the shorter piers near the abutments. Further, pier forces, even in the regular central portion of the bridge would be significantly reduced by seismic isolation. A fully isolated solution has been adopted for the repair/retrofit final design.

A study on the effects of vertical input ground motion and of axial force variations in the friction pendulum isolation units confirmed that there is a general potential for significant increment in shear and bending moment demand at the pier top, but luckily for the case of the Bolu Viaduct these effects have little influence on the critical actions at the base of the piers, essentially because of the dominant actions resulting from the large mass of the piers.

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ACKNOWLEDMENTS

The authors were contracted by the Italian firm Astaldi S.p.a. to develop the conceptual design for reinstatement of the bridge presented in this paper. The assistance of technicians employed by Astaldi, in providing all needed technical details is gratefully acknowledged. The development of the project was enhanced by useful discussions with technicians employed by the Turkish Minister of Transportation (KGM, Owner), Yuksel – Rendell (Project Engineer), Technital S.p.a (Original Designer) and Freyssinet (sub-contractor of Astaldi). The cooperation of Ing. Matteo Moratti in the final detailed design and of Ing. Paola Ceresa in the simulation of the viaduct response is also appreciated.