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The dynamic intelligent bridge:
A new concept in bridge dynamics

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ABSTRACT:

A method is put forward for designing bridges with improved performance under extreme dynamic loadings, such as strong earthquakes. The basic idea is that varying the boundary conditions can lead to an improved structural performance under dynamic actions. The specific goal is to substitute current bridge joints that have a fixed width with variable-width joints, which initially can be either closed or open depending on their length and the serviceability requirements, while under seismic loading their width is optimised either with a one-off adjustment, or continuously varying through semi-active control. In all cases, a novel device is used that permits this improved behaviour of the joints, the moveable shear key (MSK), a device for blocking the movement of the bridge deck, which is not permanently fixed to the seat of the abutment but can slide, hence opening a previously closed gap or closing an existing gap between the deck and the abutment. The performance sought by varying the joint gap depends on the design objectives. A pilot study on the effect of gap size is also presented, which illustrates that it can significantly affect the response quantities of the abutments.

Keywords: Bridge design, seismic loading, boundary nonlinearity, semi-active control, movable shear keys

1. INTRODUCTION

The paper presents the concept of the ‘dynamic intelligent’ (DI) bridge that has improved performance under extreme dynamic loadings, such as strong earthquake or strong winds-hurricanes. The basic idea is that varying the boundary conditions can lead to an improved structural performance under different dynamic actions. The key idea is the control of joint opening through a special type of moveable shear key (MSK) that can be controlled in a number of alternative ways, ranging from simple rough interfaces to semi-active control. The bridge behaves in the most favourable way under dynamic loads due to the presence of joints that can open in either direction (longitudinal or transverse).

Boundary conditions play a key role in the response of bridges to different kinds of loads. Support conditions in bridges are relatively easy to control, as different types of bearings and shear keys that block or release various degrees of freedom are currently available. In the case of environmental loads, the key role of the longitudinal joints is to reduce stresses generated from expansion and contraction due to temperature and time-dependent effects like shrinkage and creep. The usual design approach for joint gaps is to accommodate the full displacement due to the permanent and quasi-permanent actions and a fraction of the displacement due to temperature variations within the end joint (e.g. BSI, 2012). The presence of longitudinal and/or transverse gaps at the ends of the bridge may result in more favourable dynamic response, depending on the intensity and frequency content of the dynamic input. Kappos & Sextos (2009) have studied the effect of boundary conditions on the seismic response of a bridge focussing on the effect of closing of gaps at the bridge ends and the changes in the seismic response of the bridge as these gaps closed at earthquake intensities higher than the design one. Neither this, nor any other study has considered alternative solutions for the design of gaps and in all relevant studies the gap size was kept constant during the analysis.

The performance of bridges under low probability loadings like large temperature variations or extreme dynamic loads can be quite poor and points to the need for a proper performance-based design. There are some spectacular cases of bridge collapses due to *extreme winds*, such as that of the Kinzua Viaduct in Pennsylvania that collapsed in 2003 due to a tornado with speeds between 73 and 112 mph, while 45 bridges were damaged during Hurricane Katrina, some of them collapsed (due to span unseating), with a total cost estimated at over $1 billion (Padgett et al., 2008). There are several examples of heavy damage or collapse of bridges from strong *earthquakes*; a well-known one is that of the Hanshin Expressway during the Kobe Earthquake due to failure of some piers (Kawashima and Unjoh, 1997), while there were several bridge collapses (Romero, Lo Echevers, La Mochita, Llacolen, Tubul) during the 2010 Chile Earthquake (Yashinsky et al., 2010).

The response of bridges to the aforementioned types of low-probability loadings (normally associated with natural hazards) can be improved by the use of *structural control*. Among the different types of control techniques available, *semi-active* control emerges as a rational combination of efficiency and cost, and was recently implemented mainly as a retrofit measure to control cable or deck vibrations in bridge structures across Asia, Europe and the United States. The specific solutions implemented so far include (Gkatzogias and Kappos, 2016): (1) Variable orifice dampers, which are devices that use a controllable, electromechanical, variable-orifice valve to alter the resistance to flow of a conventional hydraulic fluid damper and hence control the damping coefficient; (2) Semi-active stiffness control devices that can vary (either continuously or on an on-off basis) the stiffness of the bracing system of the structure; (3) Friction controlled devices, either in the form of friction dampers (energy dissipators), or as components within sliding isolation systems (coefficient of friction at the sliding bearing interface controlled by adjusting the fluid pressure inside the bearings); (4) Controllable fluid dampers that use controllable fluids instead of electrically controlled valves or mechanisms used in passive fluid dampers; (5) Controllable tuned mass and liquid dampers that consist, in general, of a secondary mass with properly tuned spring and damping elements, providing a frequency-dependent hysteresis that increases damping in the primary structure; (6) Negative stiffness devices which exhibit hysteresis loops with negative stiffness to prevent the transfer of large damping forces developed in long-period base isolated structures with high values of damping ratios to the main structure while maintaining large energy dissipation. All these devices (described in detail in Gkatzogias and Kappos, 2016) are typically in the form of dampers or, less often, as braces connected to, or components of, the sliding isolation system.

This paper describes the basic concepts in designing a dynamic intelligent bridge, as well as a pilot study exploring the influence of varying joint gaps on the seismic performance of a typical bridge. If the proposed concept proves to be effective, i.e., if the additional cost required for installing the devices for controlling the opening of the joints is found to be outweighed by the reduced cost of damage due to dynamic loading, then one can claim that a new improved type of bridges can be constructed, with a reduced life-cycle cost.

2. OVERVIEW OF THE RPOPOSED SYSTEM

The proposed novel device that permits varying the joint gap, the MSK, is a stopper, arranged usually internally (Fig. 1b), but in the transverse direction, can also be placed externally, to the deck (Fig. 1a), which (unlike currently used shear keys) is not permanently fixed to the seat of the abutment but can slide, opening a previously closed gap or closing an existing gap between the deck and the substructure. Vertical support to the deck during the lateral displacement is provided either by the widely used in bridges system of elastomeric bearings or, preferably, by friction pendulum bearings that have the advantage that they restore the bridge to its initial position when the action causing the horizontal displacement ceases.

2.1 System performance requirements

The performance sought by varying the joint width (gap) depends on the key design objectives of the structural design, i.e.:

(1) When the durability of the bridge and the cost of maintenance are the key considerations, MSKs are provided in the longitudinal direction of the bridge, at one abutment only (Fig. 1c), while the other one is monolithically connected to the deck (fully integral). The gap remains closed under normal

temperature variations and contractions caused by shrinkage, creep and prestress in concrete decks. The key goal is to further extend the range of spans permitted using the aforementioned techniques by allowing a gap to form at the end of the bridge where the MSKs are located when environmental actions exceed a predefined limit. More details for this solution, which is not the focus of the present paper, can be found in Kappos (2016).

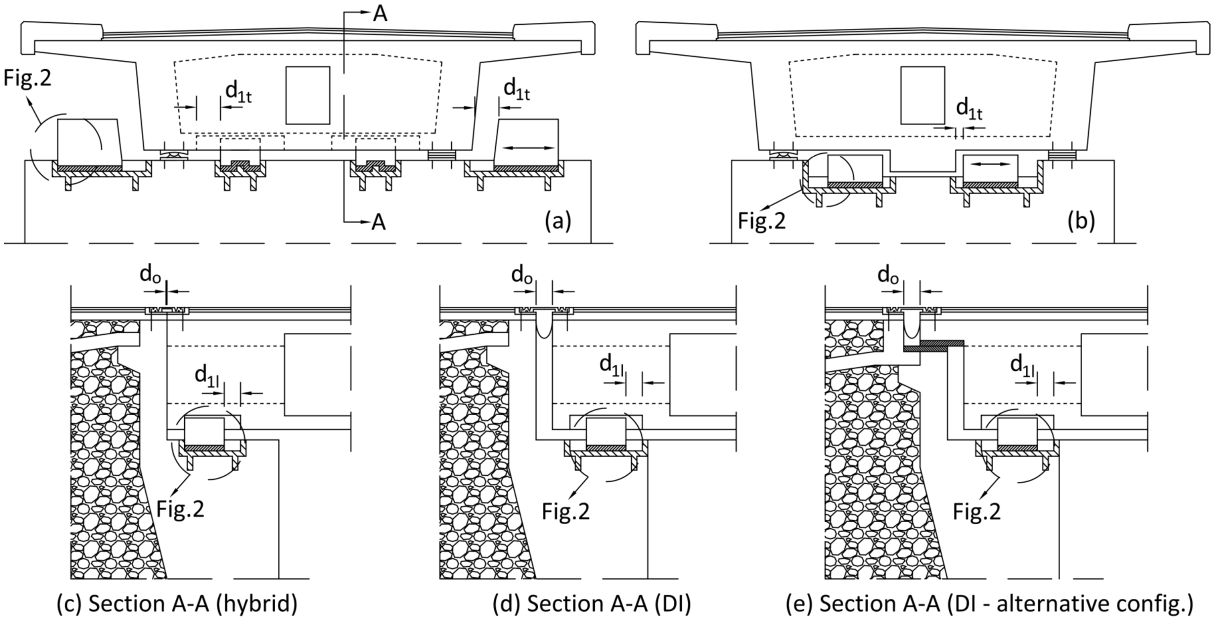


Figure 1: Possible configurations of bridges with adaptive boundary conditions: moveable shear keys in the transverse direction (top); in the longitudinal direction (bottom)

(2) In the case that low-probability, high-amplitude, dynamic loads (e.g. strong earthquakes or hurricanes) are a key consideration, an open gap in the transverse direction of the bridge (Fig. 1a) reduces the stiffness and results in generally more favourable behaviour under, e.g. medium-intensity earthquakes, whereas a closed gap is preferable for strong earthquakes under which control of displacements and prevention of unseating are the primary performance requirements. The longitudinal gap is also variable, as described in case (1).

Two different solutions are put forward in this case: (i) An *adaptive passive* system, wherein the optimum gap size is applied to the bridge by displacing the shear keys, as soon as an early warning system and/or a measure of ground acceleration in the area of the bridge is transmitted to the actuators of the bridge control system. (ii) A *semi-active* system (primarily meant for major bridges) that entails closed-loop feedback control wherein the position of the shear key can be adjusted in (almost) real-time to obtain the optimum dynamic response of the bridge.

2.2 Feasibility and theoretical framework

The key issue in the dynamic intelligent bridge (DIB) is to adjust (through the MSKs) the joint gap size in a way that the stiffness of the bridge is optimised with respect to the frequency content of the dynamic loading. The proposed MSKs might be either 'adaptive passive' (no change of device properties during the excitation of the bridge) or semi-active (variation of device properties based on controlling specific response parameters). In the first case the MSK is binary (on-off), in the second it is continuously moveable (resulting in a variable width of the joint to which it is attached). The semi-active solution is primarily meant for the dynamic control of major bridges, due to the higher costs associated with installing and maintaining the control devices; in this case movement of the MSK can only be achieved by a piston or equivalent mechanism, which is feasible but clearly increases the cost of the device. A preliminary study of gap size is presented in the next section.

In the case of the adaptive passive system, the optimum (in a practical context) gap size can be applied to the bridge by appropriate displacing of the shear keys, as soon as the early warning system and/or a measure of ground acceleration in the area of the bridge is transmitted to the actuators of the bridge control system. The "optimum design" of the devices is formulated along the lines: given a bridge structure with certain properties (defined deterministically or as random variables) and a particular intensity/frequency content of the earthquake, what would be the optimal shear key positions such that a particular performance is achieved? This problem needs to be solved "off-line" using classical optimum passive control approaches to provide basic parametric "design charts" from which "optimal" shear key properties are determined for a given bridge (system) and earthquake (input) parameters (or earthquake scenarios) and for a given performance index. In principle, the "adaptive passive" case would be easier and cheaper to implement in practice but requires more off-line "design" work.

The case semi-active system entails closed-loop feedback control in which the position of the shear key can be adjusted in (almost) real-time (i.e. with some minor delay). This involves the regulation of the electromagnetic force/current, continuously or in discrete steps. Various algorithms can be used to achieve effective regulation, e.g. based on classical, optimal, adaptive or model-predictive control. The overall objective of the control design is to achieve an acceptable level of dynamic response, subject to realistic control energy and actuation displacement constraints. Several challenges arise in attempting to analyse the dynamic response of the DIB; for instance, all existing software packages can only deal with link/gap elements with a constant gap size; hence they have to be extended to accommodate gap elements whose size will be continuously updated based on the output of the (semi-active) controller. The controller may be either tuned to the main characteristic frequencies of the bridge or operate over a broader bandwidth. In the case of optimal control, various well-tested design methodologies can be applied to design the controller, e.g. time-domain optimisation based on the maximum principle, LQG or H-infinity optimal control. Whatever method is chosen, the control solution should be robust, i.e. insensitive to unknown spectral characteristics of the seismic signal (which are likely to arise) and to model uncertainty, both parametric and unstructured (e.g. uncertainty arising due to the presence of high-frequency modes that have been ignored, mode interaction, etc.). The choice of appropriate sensors (accelerometers, displacement sensors) and their location is also an important aspect of the control design, along with the effective estimation of dynamic states and parameters, which cannot be directly measured. Finally, the effects of nonlinearities, actuation delays and force saturation constraints need to be fully taken into account during the design and control validation stages, which involve extensive simulation work. Overall, this is a more expensive solution compared to the adaptive passive one, but it requires less work at design stage and is "smart" (i.e. self-adjusted).

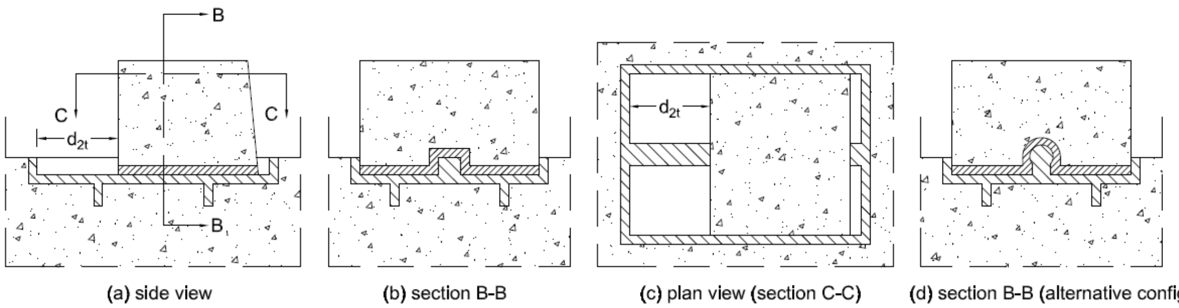


Figure 2: Views of alternative shear key configurations

Contact interaction (boundary nonlinearity) affects drastically the dynamic/seismic response of both the deck and the MSKs. Given the high mass of the deck, the contact forces/impulses imparted on MSKs, in both directions of contact (normal and tangential) are significant, and should be accounted for. A particular issue to be addressed is to simulate the dynamic interaction between the deck and the shear keys, or between the shear keys and the steel stoppers (Fig. 2). A nonlinear dynamic response analysis scheme has to be developed that is capable of capturing the dynamic response of bridges with contact phenomena. The envisaged simulation hinges on the (kinematical) impenetrability constraint between the contacting surfaces and offers a refined treatment of the (frictional) contact interaction. Aim of the simulation is to capture the non-smooth and nonlinear dynamic response of the bridge, and to assess the tendency (if any) of the MSKs to topple and/or twist as a result of contact/impact. The local effects of contact (e.g. the plastic deformation around the interface, shear fracture of the shear key, yielding of the steel stopper or the rail) have to be examined numerically, deploying spatially localised, detailed, nonlinear three-dimensional finite element simulations of the MSK sub-system using explicit time-integration. The numerical simulation of the MSK response is particularly challenging when it is supported on electro-magnetically connected steel plates.

3. PILOT STUDY OF THE EFFECT OF GAP SIZE

As a first pilot study to check the feasibility of the novel concepts described in section 2, the effect of varying the design gap size in an actual bridge on the key response quantities was studied. This sort of analysis will reveal the effect that the gap size can have on response quantities of a bridge such as deck displacements and shear forces in the piers and the abutments.

3.1 Description and modelling of the selected bridge

The selected structure (Overpass T7 in Egnatia Motorway, see Fig. 3), is of a type common in modern motorway construction in Europe. The 3-span structure with total length equal to 99 m is characterised by a significant longitudinal slope (approximately 7%) of the 10 m wide prestressed concrete box girder deck that results in two single column piers (cylindrical cross section) of unequal height (clear height of 5.9 and 7.9 m). The deck is monolithically connected to the piers, while it rests on its abutments through elastomeric bearings. Horizontal movement in both directions is initially allowed at the abutments, while longitudinal and transverse displacements are restrained whenever a 100 and a 150 mm gap (between the deck and the abutment) is closed, respectively. The bridge rests on firm soil and the piers and abutments are supported on surface foundations (footings) of similar configuration. The above geometrical characteristics (i.e. different pier heights and unrestrained response of the deck at the abutments) result in an increased contribution of the second mode. The bridge generally conforms with EC8 requirements, for a design PGA of 0.16g (return period T_r=475 yrs.) and subsoil class ‘C’, for bridges with ductile behaviour.

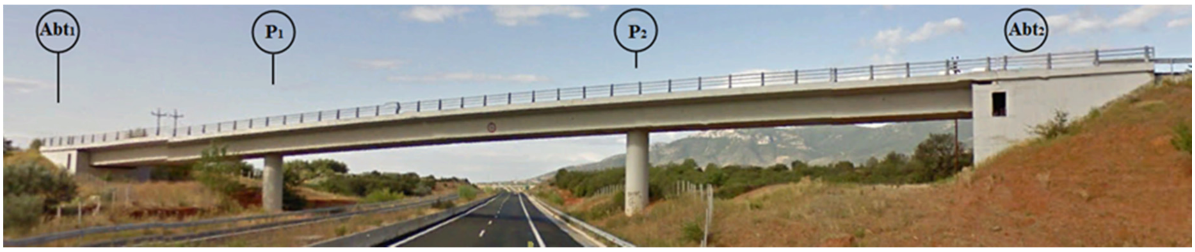
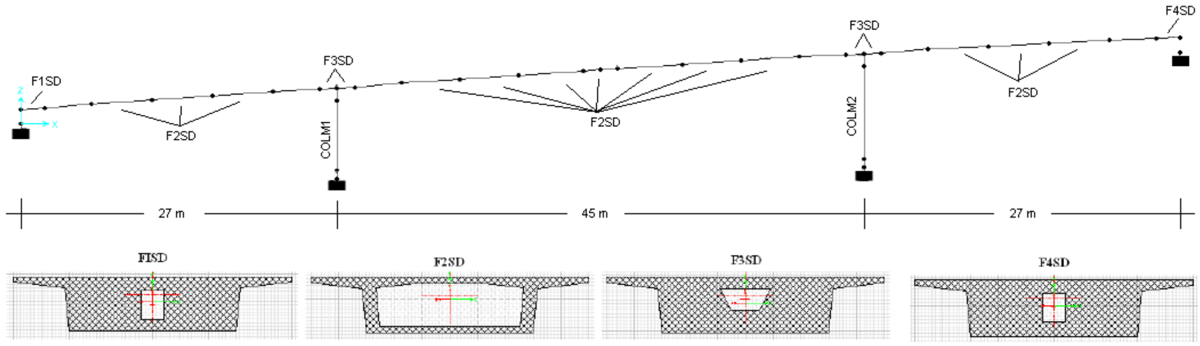


Figure 3: Studied bridge (Overpass T7, Egnatia Motorway, N. Greece)

The analysis of the bridge was carried out using SAP 2000 (CSI 2006); the reference finite element model (Fig. 4) involved 32 non-prismatic 3D beam elements. The elastomeric bearings present at the abutments were modelled using equivalent linear springs (‘Link elements’ in SAP2000 with 6 degrees of freedom). For modelling the closing of the end joints in either direction, gap elements connected to the deck through rigid elements were used (Fig. 5). The size of each gap element corresponds to the actual sizes i.e., 100 mm for the longitudinal gap and 150 mm for the transverse gap. The stiffness of the gap element is assumed 0 as long as the gap is open. Once the displacement in each direction exceeds the gap size, the stiffness will suddenly increase to that of the abutment/backfill system. The latter was estimated using the simplified procedures (based on the limited available test results) adopted by Caltrans (2013), wherein the longitudinal stiffness can be calculated from the initial embankment fill stiffness Ki ≈28.7 kN/mm/(m width of the wall), and this has to be adjusted proportionally to the backwall height. In the transverse direction, a nominal abutment stiffness equal to 50% of the elastic transverse stiffness of the adjacent bent was used (Caltrans 2013). This nominal stiffness has no direct correlation or relevance to the actual residual stiffness but is meant to suppress unrealistic response modes associated with a completely released end condition. In the case studied here, this assumption leads to underestimation of the actual stiffness of the bridge in the transverse direction.

The ground motions considered for the pilot study were five artificial, spectrum compatible accelerograms, which matched the Eurocode 8 design spectrum for the aforementioned PGA and ground conditions. Due to the good match, the variability in the calculated results was quite low.



Parameter	F1SD	F2SD	F3SD	F4SD
A (m^2)	12,38	5,89	12,35	12,81
I_x (m^4)	12,28	8,41	12,63	13,54
I_y (m^4)	4,47	3,34	4,74	4,97
I_z (m^4)	56,37	40,73	56,99	57,72

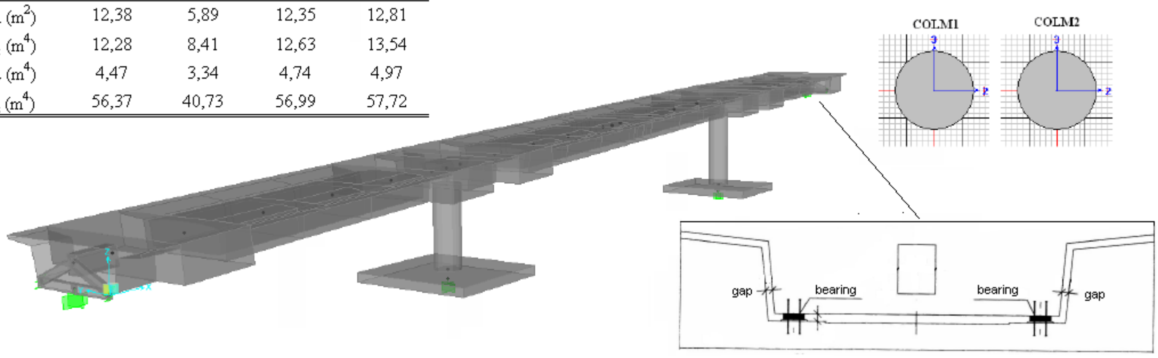


Figure 4: Layout of the bridge configuration and finite element modelling.

Three different ‘scenarios’ were considered regarding the gap sizes, involving values equal to the design ones (actual bridge), one half the design ones, and twice the design ones. It is worth recalling here that the exact size of the joint gaps depends, to a certain extent, on the judgement of the designer, especially as far as the maximum size is concerned.

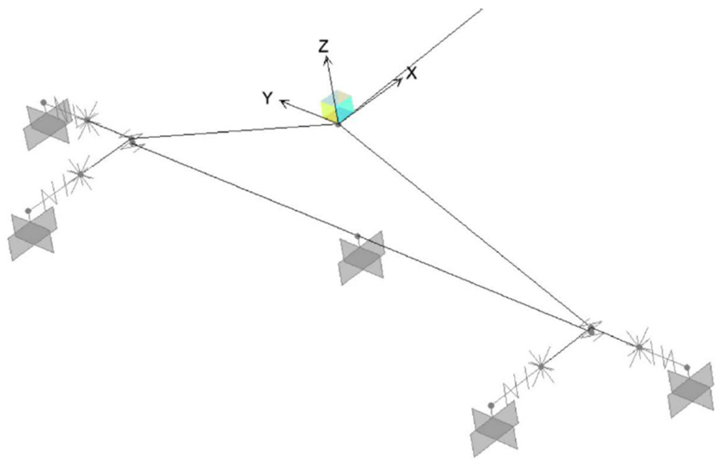


Figure 5: Modelling of abutment area, including gap elements.

3.2 Preliminary results from the analyses

Linear response-history analyses for the 5 accelerograms appropriately scaled to match first the design spectrum and then twice that intensity (corresponding to an earthquake with a return period of at least 2500 yrs.), showed that the 3rd scenario (gaps equal to twice the value in the actual bridge) was not particularly helpful in assessing the effect of the gap size, as the gaps remained essentially open in both directions even for the 2500 yr. earthquake. For the other scenario involving half the design gap size(i.e. 75 mm), there was closing of the gap in the transverse direction at Abutment 2 (Fig. 2) for the design earthquake and closing of both gaps for twice the design intensity. On the other hand, the longitudinal gap was found to remain open even for the latter case; it is recalled here that longitudinal gaps do not accommodate only the seismic displacement but also that due to temperature variations, creep, shrinkage and prestressing of the deck.

Table 1 summarises the key results (pier moments and shears, and abutment shears) for the other scenario involving half the design gap size and the standard case of the as-built bridge. It is noted that the maximum transverse displacement at the abutments is 152 mm in the as-built bridge (i.e. just exceeding the design gap size) and 98 mm in the half-gap size scenario, i.e. well above the assumed gap size, the displacement beyond the 75 mm being due to the deformability of the backfill system. It is clear from these results that while response quantities like pier moments are little affected by the gap size, other quantities, in particular shears in the abutments are very much affected (they are 2.0 to 2.5 times higher when the transverse gap is equal to one half the design value). Conversely, the displacements of the bridge are smaller when the gap is smaller than the design value.

Table 1. Key response quantities for bridge subjected to twice the design earthquake intensity.

Response quantities	Member	original gap size	half gap size
Shear forces (kN)	Pier 1	11373.6	10635.1
	Pier 2	7910.7	7539.1
	Abutment 1	1406.1	3472.3
	Abutment 2	1579.7	3306.1
Bending moments (kN.m)	Pier 1	54704.1	55469.1
	Pier 2	48571.2	48658.3

It has to be particularly noted that the reported results refer to elastic behaviour of all bridge components (including the piers), hence the only nonlinearity is due to the opening and closing of the gaps. It is clearly important to extend the scope of response-history analysis by introducing nonlinear response of the piers, and even the abutments; this work is currently under way.

4. CONCLUDING REMARKS

A new method was presented for designing bridges with improved seismic performance. The basic idea is that varying the boundary conditions through the use of the proposed novel devices called moveable shear keys (MSK)s, can lead to an improved structural performance under seismic actions of different levels, including those higher than the design ones. The envisaged variable-width joints are initially either closed or open depending on the bridge length and the serviceability requirements, while under seismic loading their width is optimised either with a one-off adjustment, or continuously varying through semi-active control. The performance sought by varying the joint gap depends on the design objectives; in the case of strong earthquakes, an open gap in the transverse direction of the bridge reduces the overall stiffness, and results in generally more favourable behaviour under, e.g. medium-intensity earthquakes (typically associated with the requirement for uninterrupted operation of the bridge), whereas a closed gap is preferable for strong earthquakes under which control of displacements and prevention of unseating are the primary performance requirements.

A pilot study on the effect of gap size was presented, involving a typical modern overpass with joints in both the longitudinal and the transverse direction. By examining different gap size 'scenarios', it was found that this size can significantly affect some key response quantities like shear in the abutments. Further studies are currently under way to assess the feasibility of the proposed novel ideas.

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