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Estimation of near-field ground motion at the Bolu Viaduct during the 1999 Düzce earthquake

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ABSTRACT

The paper revisits the 12th November 1999 earthquake in Turkey attempting to derive realistic estimates of ground motions experienced near the site of the severely damaged Bolu twin viaduct. The novelty of the research lies on the fact that the proposed ground motions are chosen on the basis of their ability to reproduce the observed rocking response of two sets of pre-cast concrete girders temporarily stored along the highway axis at the time of the event –a case study that has hitherto received scarce attention in literature. Although lying in literally adjacent locations, all girders of the first set toppled while most girders of the second set, with a different orientation than the first, remained standing. Scenarios of hybrid ground motions that account for likely near-fault effects are devised and calibrated with nonlinear finite-element analyses. It is shown that the most likely motions that can simultaneously provoke toppling of the girders located on the first site and non-toppling on the neighboring site are those combining a forward-rupture directivity pulse having amplitude between 0.25 g and 0.30 g in the fault-normal direction, with a fling-step pulse in the fault-parallel direction that is compatible with a dislocation exceeding 60 cm.

Introduction

This paper revisits the 12 November 1999 Düzce earthquake event that caused nearly 1000 fatalities, 5000 injuries and damage beyond repair to a huge number of multistory buildings. From an engineering viewpoint, the event was undoubtedly marked by the failure of the Bolu Viaduct (part of the Trans-European Motorway-TEM) that links Ankara to Europe. By the time of the event the Viaduct was a newly built structure designed according to the AASHTO Standard Specifications (AASHTO 1992) for a 500-year return period with design peak ground acceleration of 0.4g. Although the design of the viaduct incorporated a seismic protection system (sliding bearing and yielding steel devices on top of each pier) accompanied by stoppers and cable restrainers at the expansion joints (*Ghasemi et al, 2000; Marioni, 2000*), the bridge suffered severe damage and only narrowly avoided collapse.

From a seismological perspective, the 12th November 1999 Düzce earthquake (M_w 7.1) was a bi-lateral predominantly strike slip event resulting in a 40-km-long surface rupture zone (Figure 1). The rupture was initiated at the eastern end of the Karadere valley, 5 km west of the Eften Lake, where it overlapped with the eastern end of the Kocaeli rupture (*Hartleb et al., 1999*), and

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evolved in an almost clear EW direction until it disappeared while approaching the Bolu Mountain area (*Akyüz et al, 2002*).

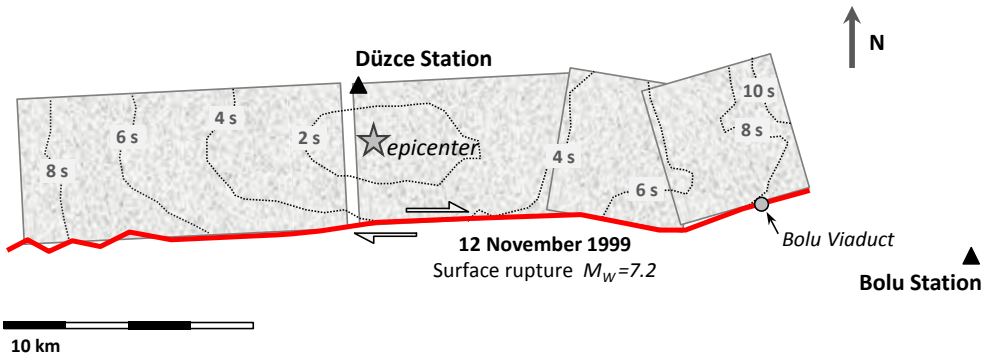

Figure 1. The Düzce 1999 fault rupture that caused the failure of the Bolu Viaduct. Isochrones show the propagation of the rupture front at 2 sec intervals (by *Konca et al, 2010*).

Judging from the specific characteristics of the rupture process it is expected that ground motion close to the surface rupture and towards the direction of slip forward propagation (i.e. within a radius of 5-10 km which encloses the Bolu Viaduct site) may either exhibit the consequences of 'forward directivity' pulse – in the normal to fault component, or may contain a significant permanent static displacement 'fling step' – in the fault-parallel direction.

Unfortunately, there were no strong ground motion recordings, either at the bridge site, or at the broader Kaynaşlı region, where the degree of experienced damage was overwhelming. The only relevant recording stations were those at Bolu (BOL) and Düzce (DZC) located at epicentral distances of 8.3 km and 20 km respectively. In a following section the two records will be assessed as to their ability to describe the intensity and the characteristics of shaking at the severely damaged zone. In addition, a set of possible excitation motions capable of reproducing the observed damage pattern at the near field region will be proposed. To this end, this paper follows a theoretical methodology to mathematically formulate hybrid motions which are in turn validated by means of back-analyzing a post-earthquake field observation which is considered as an invaluable assessment tool that has so far received limited or no attention. The case study refers to the rocking response (distinguished between toppling or non-toppling) of two sets of pre-cast concrete girders lying in the immediate vicinity of the damaged viaduct at the time of the earthquake event.

Documentation of the Reference Site

The site of the failed viaduct was documented immediately after the earthquake during the Reconnaissance Visit of the National Technical University of Athens (NTUA) Soil Dynamics Group in December of 1999. The present study focuses on the areas marked as A and B on Figure 2(a), located at a distance of about 1 km from the Bolu Viaduct. Area A covers a length of approximately 200 m, and is traced in a highway section that advances in a SE25^o direction, while the starting point of the neighboring area B is located immediately after a slight left turn of the highway, along a section with an almost E-W orientation (Figure 2b). At the time of the earthquake, areas A and B were occupied by temporarily stored precast concrete girders to be used for the construction of the second viaduct nearby. The girders were 40m in length while

their cross-sectional outline had a trapezoidal shape**.** Each girder was not lying directly on the highway embankment but was supported on two square concrete pads located at the two ends.

Surprisingly, during the Düzce 1999 event the two sets of girders (i.e. those placed at Area A and B respectively) responded in a completely different manner: *all girders of Area A toppled towards the same direction, while the girders of Area B remained intact in their upright position* (Figure 2c). Since the closest distance between the two sets of girders was of the order of few meters, it would be erroneous to explain the different response calling upon factors related to local site effects (i.e. difference in the soil profile or in the excitation motion). On the other hand, the slight difference in the orientation of the two sets of girders could be responsible for their experiencing different components of the same ground motion. As such, the hybrid motions that will be presented in the subsequent sections should simultaneously ensure toppling of the Area A girders and stability of those in Area B.

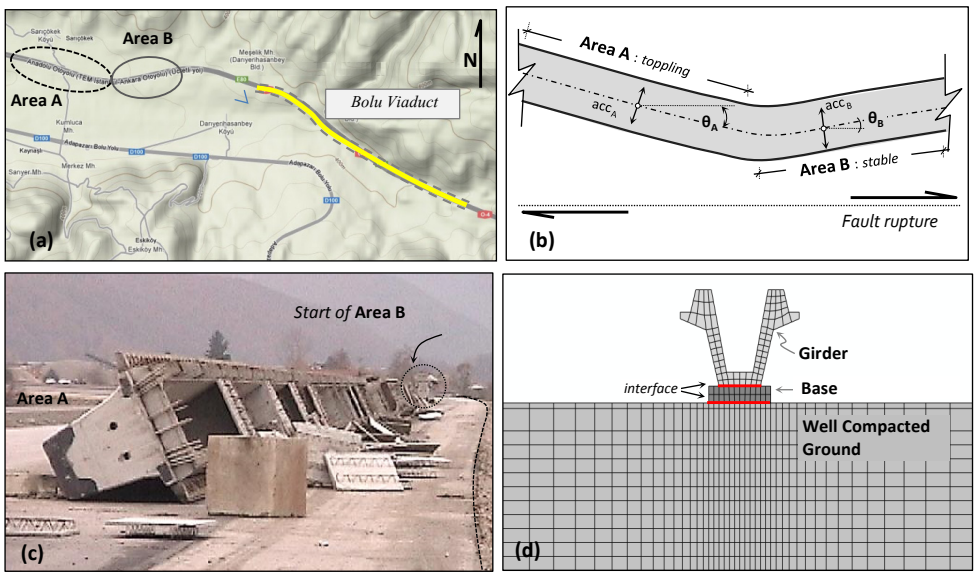

Figure 2. (a) Closer view of the two locations of interest (Areas A and B) where evidence of girder toppling and non-toppling had been observed; (b) a schematic explanation of the orientation of Areas A and B with respect to the fault axis; (c) photo of the overturned girders in Area A; (d) view of the FE mesh.

Problem Set up and Numerical Analysis Methodology

The problem is analyzed by means of the finite element (FE) method assuming plane-strain conditions, with due consideration to geometric nonlinearities (*uplifting* and *P-Δ effects*) adopting a friction coefficient of $\mu = 0.7$ between the concrete and the soil which impedes sliding (i.e. no s slippage was observed in any of the girders). Quadrilateral continuum elements (Figure 2d) have been used for the modeling of soil and girders which are connected through special purpose interface elements to model detachment and/or sliding. These elements are infinitely stiff in compression, are tensionless allowing separation. In the horizontal direction, they follow Coulomb's friction law, allowing for sliding when the friction force is exceeded. In the analyses presented herein, a friction coefficient of μ=0.7 is assumed.

The dynamic response of the system is simulated employing nonlinear time history analysis, applying the excitation at the base of the model. The FE mesh has a length of L = 25 m so as to quarantine parasitic wave reflections at its lateral boundaries while similarly, appropriate dashpot elements have been used at the bottom boundary to account for radiation damping. Additionally, free-field boundaries responding as shear beams are placed at the two lateral boundaries of the model, to simulate the motion produced by in–plane vertically incident SV waves. To account in our 2-d model for the two isolated supports at the base of the girder, the relative stiffness of the girder, the base and the soil stratum in the out-of-plane direction is properly adjusted. The numerical methodology had been tested against the closed-form solutions of *Zhang & Makris (2001)* for the simplest case of a free-standing rigid block lying on a completely rigid base subjected to trigonometric (sine and cosine pulses) base excitation (*Gelagoti, 2013*).

Assessment of the available records

Two strong motion records were available in the vicinity of this case study. Their representativeness of the real motion at the Bolu viaduct site is judged herein on the basis of their ability to explain the overturning behavior of the two sets of bridge girders and the results are plotted in Figure 3. It is clear that none of the two original records could explain the toppling of the girders in site A. The maximum developed rotation reaches a mere 0.2 rad, a value well below the critical toppling threshold.

Besides, the original records need to be amplified by an almost 50% in order to initiate overturning. This would result in extremely high acceleration levels that are neither compatible with the seismic signature of the two Turkey events, nor with the overall level of damage in towns nearby. The high-frequency BOL record needs to be scaled up to a PGA of 1.2g producing an unrealistically high spectral acceleration of 3 g at the period range of T = 0.3 – 0.5 sec. The much lower frequency DZC record needs to be amplified by a factor of 1.5 which on the one hand yields a somehow more realistic PGA value of 0.68g, but is accompanied by spectral accelerations of 2g at most periods in the region of 1 s.

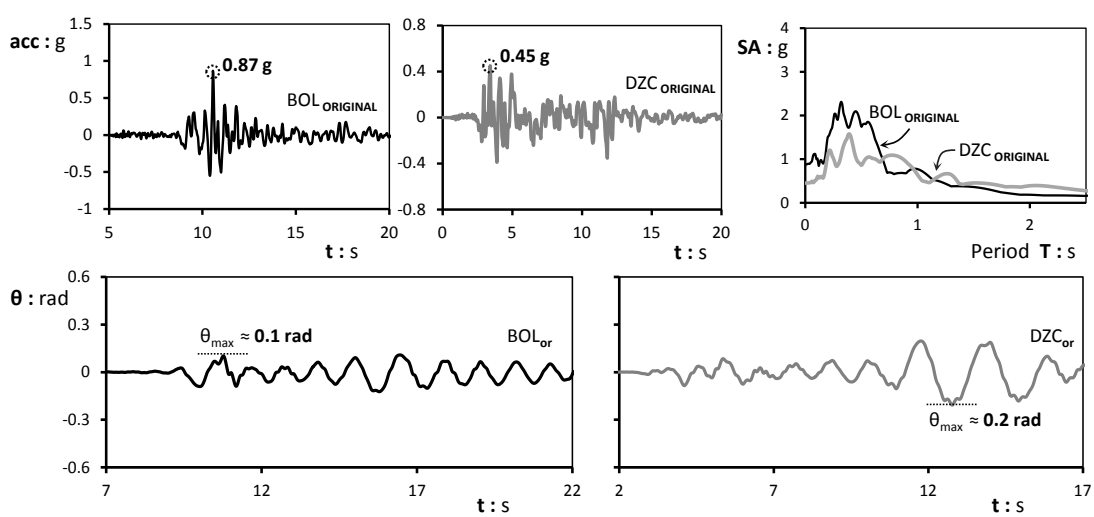

Figure 3. Bridge girders are excited by the Bolu and Duzce records : original records (time-histories and elastic response spectra) and the computed girder rotation.

Thus, the search for an appropriate ground motion calls for the derivation of a hybrid motion that will contain kinematic pulses representative of near-field motions, while simultaneously being compatible with the specific characteristics of the particular seismic event (source-site path, local site conditions etc.) in the way the latter are reflected on the high-frequency part of the recorded motions.

Formulation of a hybrid near-field record

The formulation of the ground motion at the reference site follows a two-step approach for both the fault normal and the fault parallel component. In the first step, the high frequency (HF) is essentially borrowed from the original record, and in the next step the latter is combined in the time domain with an appropriate pulse-type motion to reproduce the synthetic near-fault record. Note that the alignment of the time histories (before their addition) may be particular crucial for the resulting ground motion. However, for brevity's sake the effect of timing is ignored herein and only one plausible synthetic motion is presented. Understandably, the above approach is hardly a robust method. A theoretically more rigorous estimation of the exact HF component at the specific location would have taken into account a plethora of additional factors related to the specific site conditions.

Admissible LOW-FREQUENCY pulse-type motion

Experience from strong motions recorded in the last three decades has shown that the kinematic characteristics of the near-fault motions are governed by a large in amplitude yet narrow band velocity and displacement pulse. For strike-slip events, forward-directivity conditions are normally polarized in the component normal to the direction of rupture — shear waves are travelling ahead of, or about simultaneously with, the rupture front thus generating a strong pulse (similar to a shock wave effect) at the beginning of the record (*Somerville et al, 1997*). The aforementioned observations are summarized in the empirical correlations (Eqs. 1 and 2) of *Bray & Rodriguez-Marek (2004)* between peak ground velocity (PGV) and the period (T_p) of the forward-directivity velocity pulse with earthquake Magnitude (M_w) and rupture distance (R):

$$ln(PGV) = 4.51 + 0.34M_W - 0.57ln(R^2 + 7^2) \quad (1)$$

$$ln(T_p) = -6.37 + 1.03M_W \quad (2)$$

Assuming M_w= 7.1 and R=1÷2 km, a PGV of around 100-110 cm/s and a T_p of around 2.6 s (f_p = 0.38 Hz) may be expected for the case-study under consideration. With respect to the parallel direction to the rupture, pulse-like motions may also be present. Being associated with permanent ground displacement due to surface fault rupture, they are described by unilateral velocity pulses which in the displacement time-history are evidenced as discrete 'fling' steps.

The selected low-frequency scenario

On the basis of the previous discussion, a fairly simple low frequency scenario that is considered to be representative of the actual strong motion at the area under investigation consist of: *a symmetrical two-sided sinus pulse (with f_p= 0.5 Hz) for the directivity impact and a one-sided*

sinus pulse of longer period (f_p = 0.3 Hz) for the fling-step component (Figure 4).

The acceleration amplitude of the forward directivity pulse (α), as well as the maximum static displacement of the fling step (δ_{max}) are not predetermined; they will rather be parametrically explored in the subsequent paragraph seeking appropriate combinations of α and δ_{max} able to simultaneously reproduce the rocking pattern observed in areas A and B.

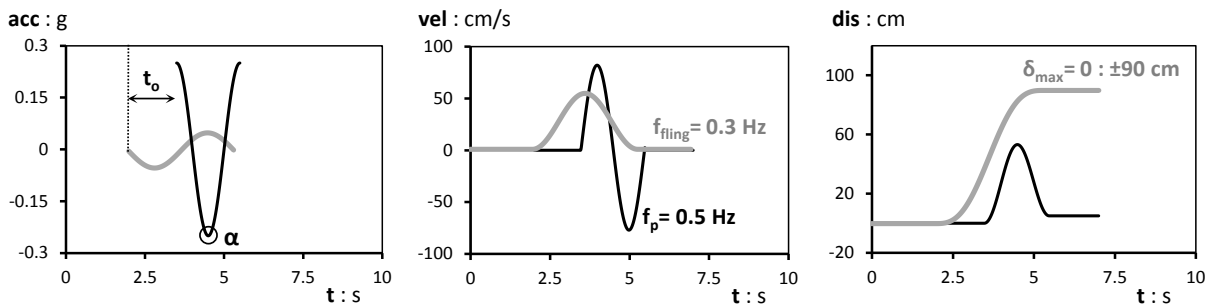

Figure 4. A possible low-frequency scenario: a sinus directivity pulse is combined with a longer-period half-sinus pulse resembling the fling step

Application to the reference site

Given the highway orientation in the area of interest, the resultant motion experienced by each set of girders (in Areas A and B) may be calculated as a function of the fault normal (acc_{NORMAL}) and fault parallel components (acc_{PAR}) of ground motion. A number of possible combinations of α (acceleration amplitude of the fault directivity pulse) and δ_{max} (maximum fault dislocation recorded in the fault-parallel time history) are tested with the intention to develop the most probable (lower-bound) estimation of the experienced motion at the reference site.

By assuming a fixed value of the fling step δ_{max} = 90 cm, for all acceleration levels between 0.25g to 0.27g , the resulting motion of Area A is adequately strong to provoke the desired toppling while at the same time the input motion of Area B remains weaker (albeit marginally) than the limiting overturning rotation (0.4 rad) ensuring that the respective girders remain standing after the shaking. Naturally, lower acceleration values (i.e. α = 0.24g) would result in a slightly weaker motion which cannot initiate toppling in the non-safe Area A. Consideration of higher acceleration levels (i.e. of α >= 0.28g), yields stronger response which is inevitably accompanied by unequivocal toppling at both areas (dashed lines). In other words, under the assumption of δ_{max} = 90 cm (and completely ignoring the contribution of the high frequency part of the motion), the admissible α values may only range between 0.25g and 0.27g.

Similarly, for a constant value of PGA fixed at 0.25 g, δ_{max} should always be higher than 60 cm to correctly reproduce the observed response. For lower δ_{max} values (i.e 20 –60 cm) the destructive interference of the fling-step and the directivity pulse is incapable of adequately reducing the motion in Area B thereby resulting in a stronger than desired motion which provokes overturning of girders in the "safe" Area.

A Probable "Hybrid" Earthquake Scenario

Based on the findings of the previous paragraph a plausible ground motion is presented in Figure 5 and compared against the originally recorded strong motion at the Düzce station (grey line). The numerical results for this admissible earthquake scenario are portrayed in Figure 6. As expected, the velocity time-histories have been drastically modified; now being characterized by a single coherent velocity pulse at the beginning of the record as opposed to a long-duration strong pulse extending along the entire time history of the original records. In terms of acceleration, the hybrid motion attains higher PGA than the critical values of the low-frequency directivity pulses identified previously (which were of the order of 0.25g) due to the superposition of the high frequency component. However, these high-frequency spikes although increasing the PGA of the hybrid motion, are not enhancing its toppling potential: a low-frequency component with amplitude as high as 0.23 g is essential for the hybrid record to initiate overturning. When compared with the original records, the resultant motions are quite similar in terms of acceleration amplitude: the hybrid motion is slightly amplified compared to the original record.

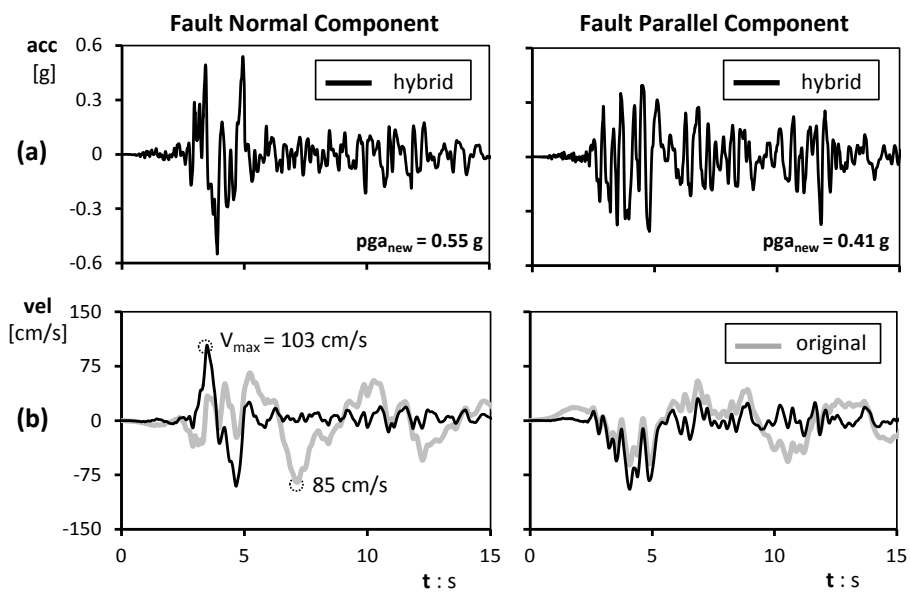

Figure 5. Admissible hybrid strong motion components (black line) assuming α=0.23g and δ=90cm and comparison with the original Düzce 1999 records (gray line): (a) acceleration, (b) velocity time-histories.

Conclusions

This paper has revisited the 12 November 1999 Düzce earthquake attempting to realistically estimate the ground motion experienced in the area of the failed Bolu viaduct. The novelty of the research presented herein lies on the fact that the proposed ground motions are validated on the basis of their ability to reproduce the observed rocking response of two sets of pre-cast concrete girders temporarily stored along the highway axis at the time of the event –a case study that has hitherto received scarce attention in literature. Consistently with published results, the examination of this case study affirms the inability of the recorded motions (at Düzce and Bolu

stations) to accurately reproduce the observed failure pattern without being unrealistically amplified. In order to mathematically formulate admissible ground motions, a two-step hybrid methodology has been proposed. The observed behavior of the two sets of girders was found to be only reproducible when accounting for their slightly different orientation with respect to the fault outbreak direction. The existence of a very significant fling-step pulse (of amplitude greater than δ = 60 cm) in the fault-parallel component of the hybrid record was found to be essential in order to provoke differences in the rocking response between the two locations. Compared with the original records, the resultant hybrid motions are quite similar in terms of PGA values but differ significantly with respect to velocity and displacement time histories.

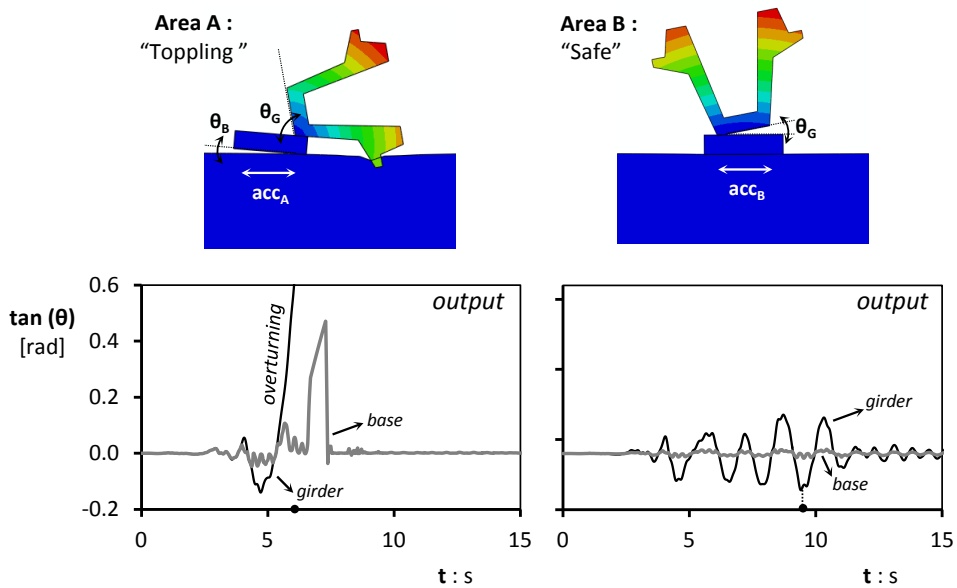

Figure 6. Numerical justification of the validity of the resultant hybrid motions : (a) snapshots of the deformed mesh at the instant of maximum girder rotation; (b) rotation time histories (black line for the girder, gray line for the underlying concrete pad).

Acknowledgments

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