Seismic Fragility of Concrete Bridges with Deck Monolithically Connected to the Piers or Supported on Elastomeric Bearings

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SUMMARY:
The seismic vulnerability of regular bridges with continuous deck, monolithically connected to the piers or supported on elastomeric bearings is studied, as a function of the bridge length, the number, section and height of piers and the number of columns per pier. At the abutments the deck is free to translate longitudinally; transversely it is either free or constrained. Prototype bridges are designed according to Eurocodes 2 and 8 (EC2 and EC8) and their fragility curves constructed as a function of peak ground acceleration. Their seismic deformation demands are estimated by 5%-damped linear analysis and their shear forces from the plastic mechanism. Bridges designed to EC8 have satisfactory fragilities; those designed to EC2 alone show remarkably good seismic performance, yet quite higher fragilities than the bridges designed to EC8. Horizontal displacements are critical for long bridges or bridges on bearings. Hollow piers are vulnerable in shear.

Keywords: bridge piers, Eurocode 8, fragility curves, RC bridges, seismic fragility

1. INTRODUCTION

Few studies exist on the seismic fragility of European bridges. Often fragility curves developed for bridges in other parts of the world are adjusted for use in Europe (e.g., Azevedo et al, 2010). Fragility curves have been produced for modern reinforced concrete (RC) bridges in Greece, based on the capacity spectrum method and adopting a default value for the standard deviation (Moschonas et al, 2009). Shear failure of the piers is generally disregarded in existing fragility studies, but is examined in a recent study of multi-span simply-supported bridges in Italy, which adopted a default value for the standard deviation and used adaptive pushover analysis of a detailed model (Cardone et al, 2011).

In this paper the seismic fragility of regular bridges with continuous deck, monolithically connected to the piers or supported on elastomeric bearings is studied. The bridges are designed according to EC2 (CEN, 2005a) and EC8 (CEN, 2005b). The results are presented as fragility curves, for which the - conditional on peak ground acceleration (PGA) - probability of the demand exceeding the capacity is calculated point-by-point. The same approach has been followed by (Franchin et al, 2006) for the fragility curves of simply-supported bridges, where the random ductility demand in the piers was obtained via incremental dynamic analysis. Herein, flexural and shear failure modes together with the deformations of the deck and the bearings define the damage states. Deterministic estimates of demand and capacity are obtained using the methods and models defined in Part 3 of EC8 for concrete buildings, appropriately adapted to bridges. The dispersion accounts for model uncertainties, the variability of materials and geometry and that of the seismic action for given PGA.

2. BRIDGE TYPOLOGIES

Prototype regular bridges with continuous deck are examined. The objective is to construct fragility curves for each specific bridge as a function of a few parameters, such as:

- deck-pier connection: monolithic or through elastomeric bearings;
- transverse translation at the abutments: free (considered here for road bridges) or constrained (considered here as more representative of railway bridges);
- bridge length *l*: 80 m, 120 m, 160 m and 240 m, as shown in Figure 2.1;
- type of pier cross-section: hollow rectangular, wall-type rectangular, circular (see Figure 2.2);
- number of columns per pier (for circular cross-sections only): 1 or 2;
- pier height *h*: 10 m or 25 m (due to slenderness limitations, h = 25 m is studied only for piers with hollow rectangular cross-section);
- level of seismic design: design for gravity loads per EC2 alone, or seismic design according to EC8 with a design PGA of 0.25g on top of Soil C, respecting also all relevant requirements of EC2.

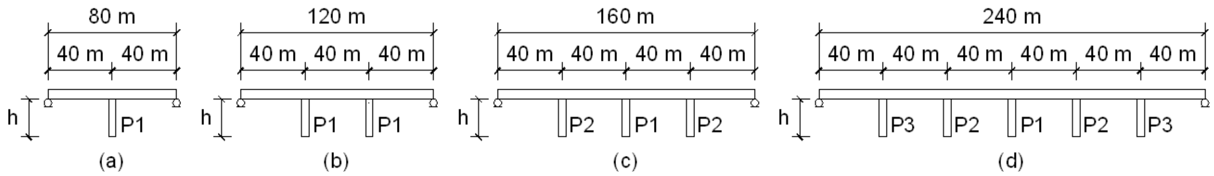

Figure 2.1. Bridges with two (a), three (b), four (c) and six spans (d)

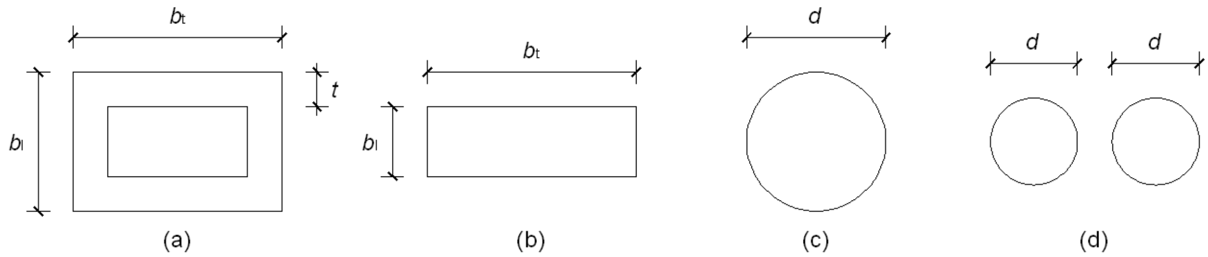

Figure 2.2. Pier cross-sections: rectangular hollow (a), wall-type (b), circular (c) and two-column circular (d)

3. DESIGN OF THE BRIDGE PIERS

The deck is a single-box prestressed concrete girder of constant cross-section, free to translate longitudinally at the abutments. For railway bridges it is taken as transversely constrained at the abutments and with a depth of 8% of the span; for road bridges it is transversely free there and with depth 4% of the span. The top slab is 14 m wide and 0.45 m thick; the bottom slab 7 m wide and 0.35 m thick. The web thickness is 0.7 m in railway bridges and 0.5 m in road ones. The deck has two 1.25 m-wide side-walks, weighing 10 kN/m each. Ballast and sleepers weigh 16 kN/m^2 and surfacing of road bridge decks 2 kN/m^2. Normal rail traffic with Load Model 71 is considered for the railway bridges and LM1 (Load Model 1) for the road ones, according to (CEN, 2003). Concrete is C30/37 and steel is of grade 500 and Class C. The prototype bridges are designed and detailed per EC2 (CEN, 2004, 2005a) and, where applicable, EC8 (CEN, 2005b).

The piers are considered fixed at the base. Their cross-sectional dimensions are chosen to meet the maximum slenderness limit of (CEN, 2004) so that 2nd-order effects may be neglected under the persistent-and-transient combination of actions per Eqs 6.10a, 6.10b in (CEN, 2002). Their longitudinal and transverse reinforcement is initially dimensioned for the ultimate limit state (ULS) in bending with axial load and shear for the above persistent-and-transient combination of actions including the effects of geometric imperfections per (CEN, 2004).

Bearings are dimensioned per (CEN, 2005e), so that in the persistent-and-transient design situation - including the effects of creep, shrinkage and design thermal actions per (CEN, 2002) - the total nominal design strain in the elastomer is $\varepsilon_{t,d} \leq 7.0/\gamma_m$ and the design shear strain due to translational movements is $\varepsilon_{q,d} \leq 1.0$. These checks are repeated in the seismic design situation, i.e. the combination of the design seismic action with creep, shrinkage and half the design thermal actions, and the bearing dimensions are updated as necessary. The seismic displacements are iteratively estimated from the elastic displacement spectrum and the stiffness of the bearings, as controlled from their size. The bearings are designed for creep and shrinkage of the deck estimated for rapid hardening cement; age at prestressing 3 days; curing for 4 days; span completion in 15 days; mean relative humidity and temperature during the lifetime 70% and 15°C; and long-term prestress balancing the permanent loads. The design uniform temperature difference from construction is ±50°C. The design of the bearings for the persistent-and-transient design situation, including creep, shrinkage and thermal actions, takes place for the combination of these actions per (CEN, 2002), considering the longitudinal displacement of the underside of the deck over the bearing due to the rotation of the deck section owing to the traffic and other vertical loads in the said combination which are not balanced by the prestress.

Seismic analysis of the bridge is performed for the EC8 design spectrum, with a design PGA on top of the rock of 0.25g and the recommended Type 1 spectrum for Ground C (firm soil). The behaviour factor is q = 1.5 for bridges with deck on bearings (limited ductile behaviour) and $q = 3.5\lambda(\alpha_s)$ for those with monolithic deck-pier connection (design for ductile behaviour), with $\lambda(\alpha_s) = \sqrt{(\alpha_s/3.0)}$ if the pier has shear span ratio, α_s, in the range $1.0 \leq \alpha_s < 3.0$, or $\lambda(\alpha_s) = 1.0$ if $3.0 \leq \alpha_s$. There is no reduction of the q -value for axial load, as the EC2 slenderness limit produces normalised axial force $\eta_k < 0.3$.

EC8's "rigid deck model" is used in the longitudinal direction of all bridges and in the transverse one of those with free transverse translation at the abutments (road bridges). It employs a single-degree-of-freedom system of the deck with the total mass of the deck plus the sum of masses in the upper half of all piers which are rigidly connected to the deck and the total stiffness of the individual supports. The effect of the rotational mass moment of inertia of the deck about its axis is considered in the transverse direction of bridges with deck monolithically connected to single-column piers, by slaving the rotational degree-of-freedom at the pier top to the horizontal translation there. If the piers support the deck via bearings, the composite pier-bearing stiffness is used and the deck displacement is attributed to the pier and the bearing in proportion to their flexibilities.

Modal response spectrum analysis is applied in the transverse direction of bridges with constrained transverse translation at the abutments, considering the deck mass as continuously distributed along its length and half of the mass of a pier as lumped at its top. In bridges with one, two or three piers (Fig. 2.1(a) to (c)), the piers are considered discretely at their exact locations. If the piers are more than three (Fig. 2.1(d)), their transverse (composite pier-bearing) stiffness is smeared along the deck and the deck is considered as on a continuous elastic lateral support.

In bridges designed for seismic action the vertical reinforcement of piers is dimensioned for the ULS in bending with axial load from the action effects from the analysis for the seismic design situation. The effective stiffness of piers for the seismic analysis is taken as in (CEN, 2005b): (a) if the pier supports the deck via bearings, the stiffness of the uncracked gross section is used; (b) if it is monolithically connected to the deck, the pier secant-to-yield-point stiffness is used, calculated from the design ultimate moment and the section's yield curvature per Annex C of (CEN, 2005b); iterations are necessary, starting from the initial reinforcement from the ULS due to geometric imperfections and the minimum requirements in (CEN 2005a, 2005b) for the number, spacing and ratio of vertical bars in the section. The pier transverse reinforcement is dimensioned to meet EC8's confinement requirements and the ULS in shear. In bridges with monolithic deck-pier connection, the capacity design shears are determined from the plastic mechanism. EC8's reduction factors for bridges with limited ductile or ductile behaviour are applied on the design shear resistance of the piers.

In two-column piers, the transverse seismic forces applied at the centroid of the deck section produce an overturning moment that induces a tensile axial load in one column and a compressive in the other. The ULS dimensioning in bending and shear and the calculation of confinement reinforcement is performed for the most adverse case of axial load (low for bending and shear, high for confinement).

The piers of bridges designed to EC8 have much higher amounts of longitudinal and transverse reinforcement than those designed to EC2, in many cases more than double. Although they have considerably higher flexural and shear capacity than those designed for gravity only, their deformation capacity is not always much higher.

4. FRAGILITY ANALYSIS AND RESULTS

Fragility curves are constructed for two damage states: yielding of the piers and ultimate condition of the piers and the bearings. PGA is selected as the intensity measure. Damage measures for the piers are the peak chord rotation at the ends where rotation is constrained and the peak shear force. The damage measure for bearings is the relative displacement between the deck and the top of the pier, which is checked against the eccentricity of vertical load that may cause rollover, and the shear deformation of the bearings. In railway bridges, the maximum angle of rotation of the ends of the deck and the peak seismic curvature of the deck, both in a horizontal plane, are checked against the operational limits set out in (CEN, 2005c).

Inelastic or elastic seismic displacement and deformation demands are estimated by 5%-damped elastic analysis, i.e., with the "equal displacement rule" (Bardakis and Fardis 2011, CEN 2005d). Elastic analyses use the "rigid deck model", except in the transverse direction of railway bridges where modal response spectrum analysis is employed. The only difference with the seismic analysis for the design of the bridge in this respect is that the unreduced 5%-damped elastic spectrum is used. The secant-to-yield-point stiffness of the piers is used (Biskinis and Fardis 2010a, CEN 2005d), with mean material strengths $f_{cm} = f_{ck}$ +8 MPa, $f_{ym} = 1.15 f_{yk}$. Following the sequence of plastic hinge formation at pier ends, the shear forces are determined from the mean values of moment resistances.

The seismic analyses give the median value of the damage measure of interest as a function of the intensity measure, i.e., the PGA of the excitation. The conditional on PGA probability of exceedance of each damage state is computed from the probability distributions of the damage measure demands (conditional on PGA) and of the corresponding capacities. The expected values of the yield and ultimate chord rotation and the shear resistance of the piers are established from the expressions in (Biskinis and Fardis, 2010a, 2010b, 2012) and (Biskinis at al 2004), which, for rectangular sections at least, have been adopted in (CEN, 2005d). Dispersions are calculated from the coefficients of variation (CoV) listed in Table 4.1 and take into account model uncertainties of demand and capacities, and the dispersion of material properties about their best estimates. The values for the pier deformation demands for given excitation spectrum are based on (Bardakis and Fardis 2011); those for creep and shrinkage on the scatter associated with the models used (CEN, 2004) and the natural dispersion of the variables they use; the CoV of daily temperature derives from a standard deviation of 10^{o}C and the presumed yearly mean of 15^{o}C. The CoVs of the RC pier capacities reflect the uncertainty in the models, including the scatter in material and geometric properties, and have been quantified in (Biskinis & Fardis, 2010a, 2010b, 2012, Biskinis et al, 2004); those for the bearings come from the literature, alongside cyclic tests on elastomeric or Lead Rubber bearings tested at the Structures Lab at the University of Patras. The same sources give the mean G-modulus and shear deformation capacity used here for such bearings, namely 900 kPa and 165%, respectively.

Table 4.1. Coefficients of variation

Demand	CoV	Capacity	CoV
Spectral value, for given PGA and fundamental period	0.25	Yield chord rotation - circular pier	0.32
		Yield chord rotation - rectangular or hollow pier	0.29
Shear force demand, for given spectral value at fundamental period	0.15	Ultimate chord rotation - circular pier	0.30
		Ultimate chord rotation - rectangular pier	0.36
Displacement demand for given spectral value at fundamental period	0.20	Ultimate chord rotation - hollow pier	0.29
		Shear resistance in diagonal tension - circular pier	0.16
Daily temperature	0.67	Shear resistance in diagonal tension - rectangular or hollow pier	0.14
Creep & shrinkage strains	0.60	Shear resistance in web compression - rectangular or hollow pier	0.18
		Deformation capacity of bearings	0.30

Fragility curves are constructed for each pier or bearing for seismic action separately in the transverse (T) and the longitudinal (L) direction of the bridge. As the analysis is deterministic, based on mean properties, the demand on a failure mode is computed on the basis of the deterministic state of the system at that value of PGA; e.g., a pier shear force is computed on the basis of the plastic hinging that has occurred with a conditional-on-PGA probability of at least 50%. The fragility curve of a component at a given damage state is taken as the worse of its possible conditions: among flexure or shear failure for the piers, or rollover and shear deformation for bearings, i.e., for perfect correlation of failure modes. The most adverse situation along the bridge is considered, wherever there is difference in the demands between piers or bearings: in the T-direction of railway bridges due to deck deflection between the constraints at the abutments; in the L-direction of bridges on bearings, due to the increase of thermal, creep and shrinkage displacements towards the abutments.

Results for bridges with monolithic connection between the deck and the piers, such as those shown indicatively in Figures 4.1 to 4.3, allow the following conclusions:

- Hollow piers without seismic design are likely to fail in shear, possibly before flexural yielding (see Figure 4.1(left)). Design to EC8 reduces very much a fragility of this type (Figure 4.1(right)).
- Piers in longer bridges are slightly less vulnerable as regards the ultimate damage state, but might have higher probability of exceeding the yielding damage state (cf. Figure 4.1(right) to Figure 4.2(left)).
- Piers in taller bridges are less vulnerable at both damage states (Figure 4.2).
- The horizontal deck rotation limit for the operation of railway bridges in (CEN, 2005c) is exceeded for PGAs of 0.1g to 0.4g; the operation limit for horizontal curvature is less critical. Although the seismic design of such bridges does not address this aspect, it improves performance in this respect. Longer bridges have higher exceedance probability of these deck deformation limits (Figure 4.3).
- In bridges with seismic design, circular and rectangular piers have similar fragilities, slightly higher than hollow ones. In bridges without seismic design, circular piers are the most vulnerable for the yielding damage state and hollow ones the least; the inverse holds for the ultimate damage state.
- Bridges with seismic design fail in the mean at high PGA levels, indicatively above 0.75g. Bridges designed per EC2 alone show some seismic resistance and are expected to survive minor earthquakes, indicatively with PGA in the order 0.15g or more in the mean.
- The longitudinal direction is the most critical for the ultimate damage state.
- Despite the large difference in their geometric and dynamic characteristics, road and railway bridges designed to the same Eurocode and PGA level have similar fragilities.

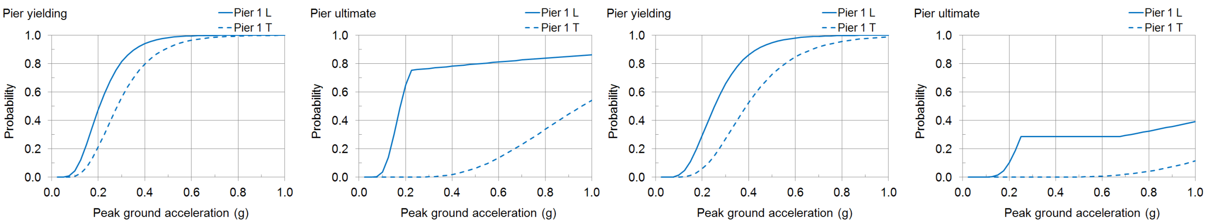

Figure 4.1. Fragility curves of road bridges with monolithic deck-pier connection, l = 80 m and hollow piers with h = 10 m, designed to EC2 (left) or EC8 (right)

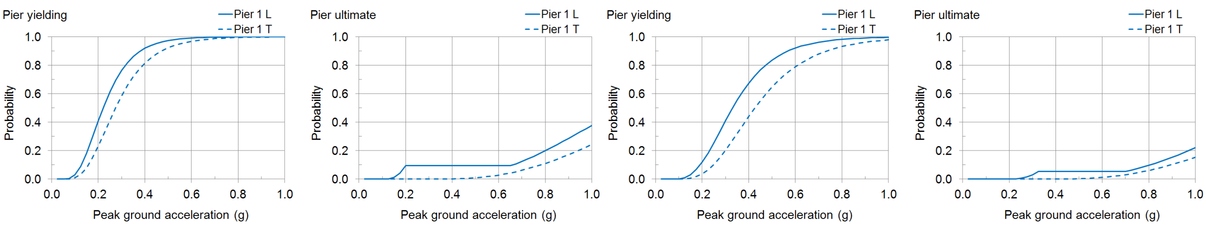

Figure 4.2. Fragility curves of road bridges with monolithic deck-pier connection, l = 120 m and hollow piers with h = 10 m (left) or h = 25 m (right), designed to EC8

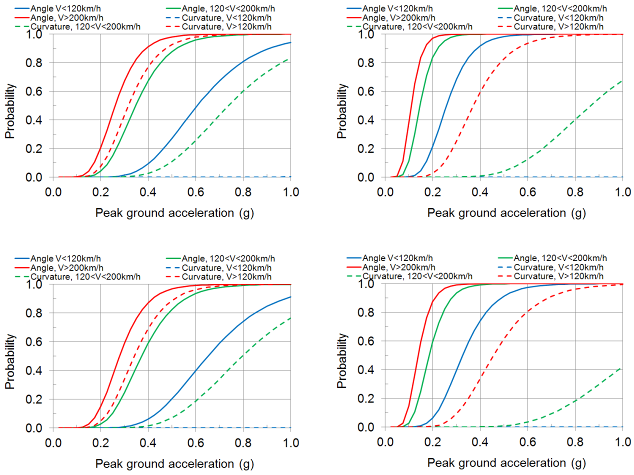

Figure 4.3. Fragility curves for the deck horizontal deformation limits of railway bridges with monolithic deck-pier connection, l = 80 m (left) or l = 240 m (right), designed to EC2 (top) or EC8 (bottom)

Figures 4.4 to 4.7 are for bridges with deck supported on bearings. The piers are numbered symmetrically with respect to deck mid-length, as in Figure 2.1. These and similar results suggest that:

- The ultimate damage state is reached because of failure of the bearings, due to rollover or shearing.
- Railway bridges designed to EC8 are less vulnerable than those designed to EC2 alone for what concerns piers and bearings, but might be slightly more vulnerable as regards the deck horizontal deformation limits (Figure 4.4).
- Piers in road bridges designed to EC8 are less vulnerable than in those designed to EC2 alone. Bearings, that are critical for the ultimate damage state show similar vulnerabilities, no matter whether the bridge is designed to EC8 or not (Figure 4.5).
- Longer and taller bridges (Figures 4.6(right) and 4.7(right), respectively) are less vulnerable as regards all damage states and damage measures.
- The type of pier cross-section does not influence the vulnerability of bridges with deck supported on bearings (cf. Figure 4.4(right) to Figure 4.6(right) and Figure 4.7(left)).
- The longitudinal direction is generally the most critical.

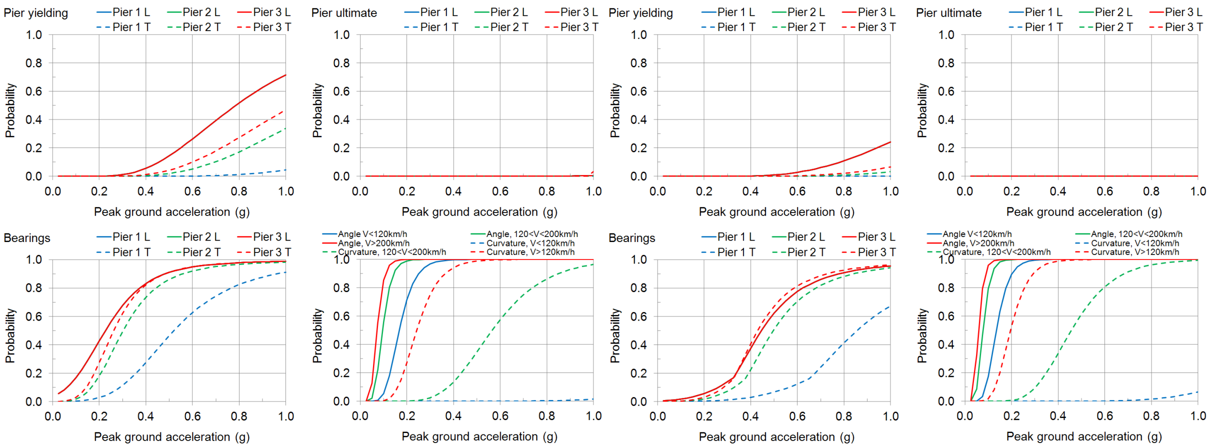

Figure 4.4. Fragility curves of railway bridges with deck supported on bearings, l = 240 m and rectangular piers with h = 10 m, designed to EC2 (left) or EC8 (right)

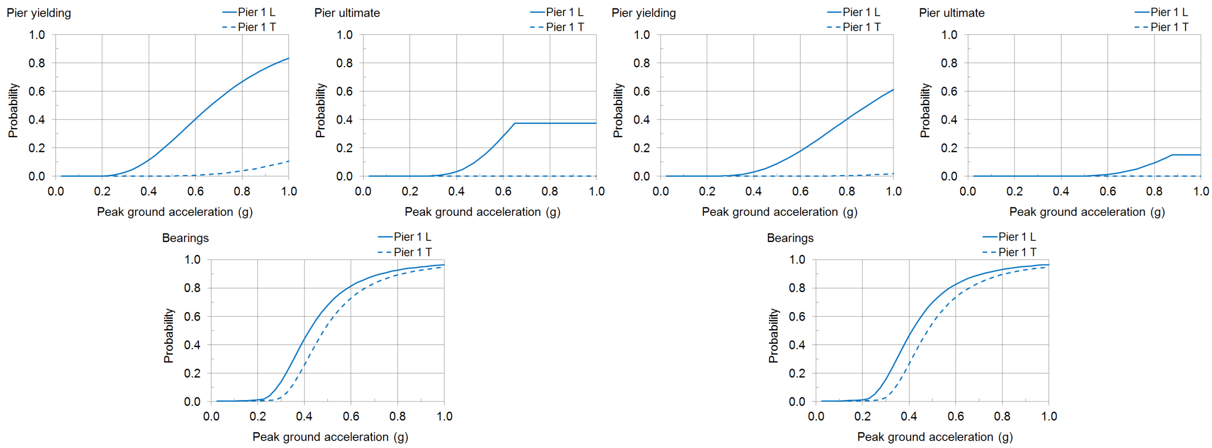

Figure 4.5. Fragility curves of road bridges with deck supported on bearings, l = 120 m and hollow piers with h = 10 m, designed to EC2 (left) or EC8 (right)

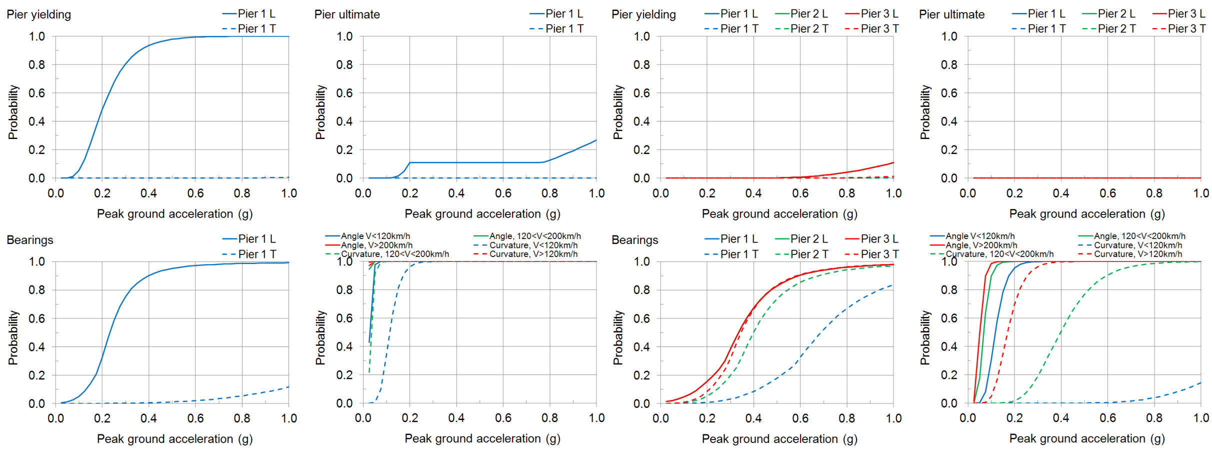

Figure 4.6. Fragility curves of railway bridges with deck supported on bearings, l = 80 m (left) or l = 240 m (right) and circular piers with h = 10 m, designed to EC8

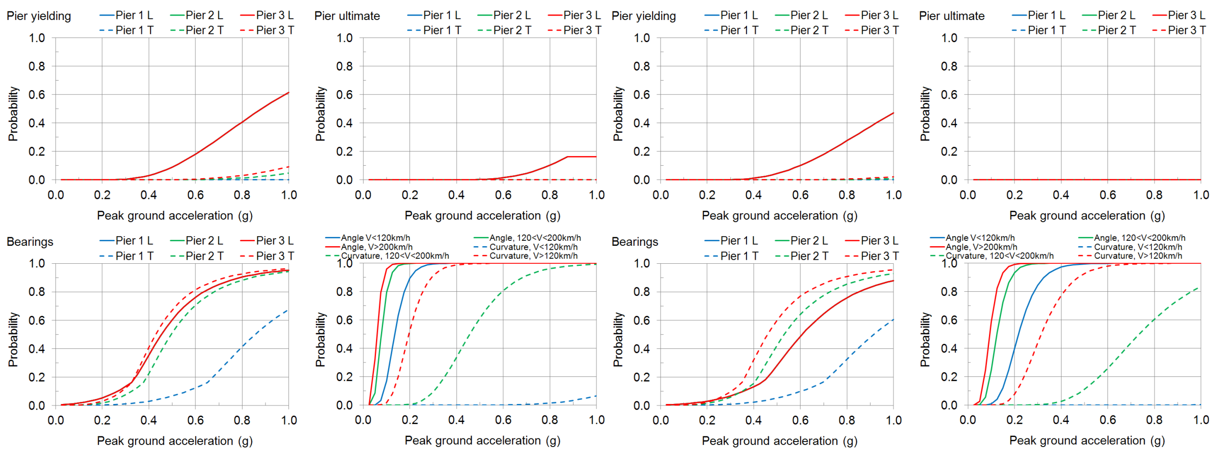

Figure 4.7. Fragility curves of railway bridges with deck supported on bearings, l = 240 m and hollow piers with h = 10 m (left) or h = 25 m (right), designed to EC8

Bridges with a fixed point at the central pier (e.g., with articulation) and the deck supported by bearings on top of the other piers are also studied. This configuration is avoided when the bridge is designed for seismic resistance; therefore only the case of design to EC2 alone is examined. The results of the vulnerability analysis lead to the following conclusions:

- Bearings are likely to exceed the ultimate limit state before the central pier (Figures 4.8 and 4.9).
- Road and railway bridges exhibit similar vulnerabilities (Figure 4.8).
- Piers and bearings in taller bridges are less vulnerable. Horizontal deck deformations are more important for taller bridges (Figure 4.9).
- Hollow piers are vulnerable in shear, but higher ones are better protected (Figure 4.9).

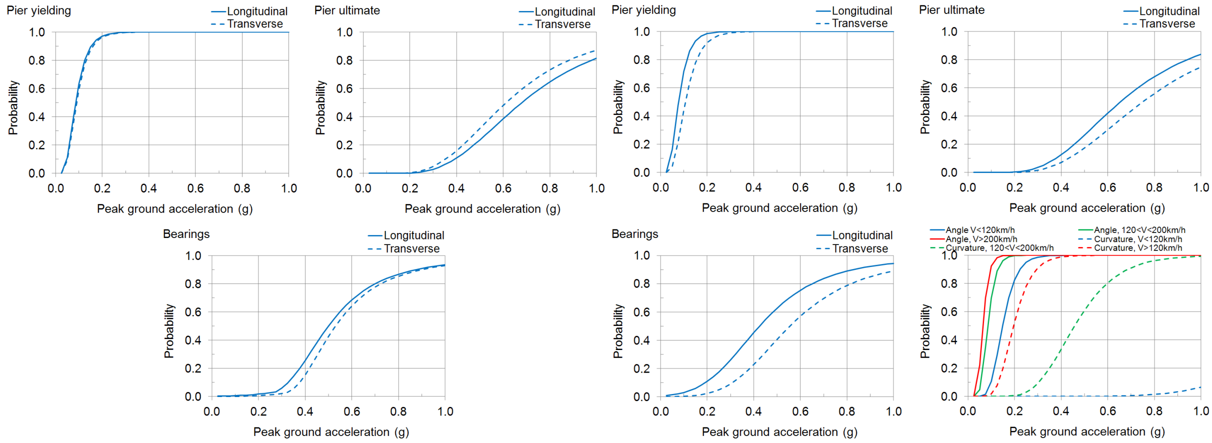

Figure 4.8. Fragility curves of road (left) and railway bridges (right) with deck supported on bearings with fixed connection to the central pier, l = 160 m and circular piers with h = 10 m, designed to EC2

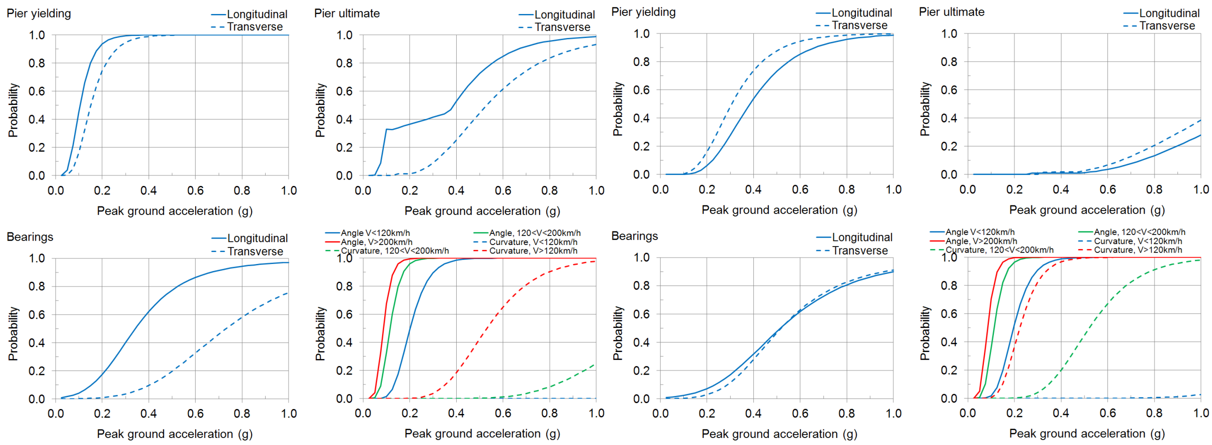

Figure 4.9. Fragility curves of railway bridges with deck supported on bearings with fixed connection to the central pier, l = 160 m and hollow piers with h = 10 m (left) or h = 25 m (right), designed to EC2

Results for the complete portfolio of bridges examined in this work are summarised in Tables 4.2 to 4.4 that list the median PGA values at the attainment of the damage states. The values for the most critical pier, direction and failure mode are given. C stands for piers with circular cross-section, H and R for those with hollow rectangular and wall-type rectangular. A dash indicates that this type of element attains this damage state in the mean for PGA > 1.0g. The data permit to extend the conclusions illustrated above to a wide range of important parameters.

Table 4.2. Median PGA (g) at attainment of the damage states in bridges deck supported on bearings and fixed connection to the central pier

			Road bridges			Railway bridges			
l (m)	h (m)	pier type	pier yielding	pier ultimate	bearing	pier yielding	pier ultimate	bearing	deck deformation
160	10	C	0.09	0.61	0.50	0.08	0.65	0.42	0.06
160	10	H	0.12	0.45	0.46	0.11	0.39	0.34	0.09
160	25	H	0.31	0.91	0.51	0.31	-	0.52	0.09
160	10	R	0.12	0.48	0.44	0.11	0.45	0.37	0.08

Table 4.3. Median PGA (g) at attainment of the damage states in road bridges

				Monolithic connection		Deck supported on bearings		
l (m)	*h* (m)	pier type	design PGA (g)	pier yielding	pier ultimate	pier yielding	pier ultimate	bearing
80	10	C	0.00	0.14	0.95	0.56	-	0.50
120	10	C	0.00	0.13	-	0.79	-	0.40
80	10	C	0.25	0.23	-	-	-	0.49
120	10	C	0.25	0.22	-	-	-	0.39
80	10	H	0.00	0.21	0.18	0.64	-	0.32
120	10	H	0.00	0.18	0.16	0.66	-	0.42
80	10	H	0.25	0.25	-	0.89	-	0.31
120	10	H	0.25	0.22	-	0.89	-	0.41
80	25	H	0.00	0.30	0.53	0.99	-	0.39
120	25	H	0.00	0.28	0.52	-	-	0.52
80	25	H	0.25	0.37	-	-	-	0.38
120	25	H	0.25	0.33	-	-	-	0.50
80	10	R	0.00	0.20	0.69	0.78	-	0.31
120	10	R	0.00	0.18	0.62	0.81	-	0.41
80	10	R	0.25	0.24	0.87	-	-	0.30
120	10	R	0.25	0.22	0.79	-	-	0.40

Table 4.4. Median PGA (g) at attainment of the damage states in railway bridges

				Monolithic connection			Deck supported on bearings			
l (m)	*h* (m)	pier type	design PGA (g)	pier yielding	pier ultimate	deck deformation	pier yielding	pier ultimate	bearing	deck deformation
80	10	C	0.00	0.13	0.95	0.26	0.48	-	0.02	0.24
120	10	C	0.00	0.13	0.95	0.13	0.62	-	0.06	0.11
160	10	C	0.00	0.13	0.74	0.11	0.78	-	0.33	0.06
240	10	C	0.00	0.10	0.51	0.11	0.75	-	0.22	0.07
80	10	C	0.25	0.19	-	0.28	0.20	-	0.23	0.01
120	10	C	0.25	0.19	-	0.15	-	-	0.37	0.05
160	10	C	0.25	0.17	-	0.12	-	-	0.37	0.05
240	10	C	0.25	0.13	0.76	0.14	-	-	0.33	0.05
80	10	H	0.00	0.17	0.16	0.37	0.44	-	0.02	0.24
120	10	H	0.00	0.17	0.16	0.26	0.55	-	0.06	0.10
160	10	H	0.00	0.17	0.16	0.24	0.66	-	0.33	0.06
240	10	H	0.00	0.17	0.16	0.32	0.64	-	0.23	0.07
80	10	H	0.25	0.21	-	0.38	0.04	0.15	0.24	0.01
120	10	H	0.25	0.21	-	0.28	0.82	-	0.38	0.05
160	10	H	0.25	0.21	-	0.25	0.87	-	0.39	0.05
240	10	H	0.25	0.20	0.99	0.34	0.88	-	0.43	0.06
80	25	H	0.00	0.27	0.92	0.29	0.58	-	0.03	0.27
120	25	H	0.00	0.27	0.92	0.17	0.66	-	0.11	0.12
160	25	H	0.00	0.27	0.92	0.15	0.80	-	0.48	0.08
240	25	H	0.00	0.27	0.92	0.18	0.78	-	0.29	0.10
80	25	H	0.25	0.32	-	0.28	-	-	0.28	0.18
120	25	H	0.25	0.32	-	0.16	-	-	0.32	0.08
160	25	H	0.25	0.32	-	0.14	0.94	-	0.49	0.08
240	25	H	0.25	0.32	-	0.17	-	-	0.46	0.09
80	10	R	0.00	0.17	0.61	0.33	0.52	-	0.02	0.24
120	10	R	0.00	0.17	0.61	0.22	0.66	-	0.06	0.10
160	10	R	0.00	0.17	0.61	0.20	0.81	-	0.33	0.06
240	10	R	0.00	0.17	0.61	0.26	0.78	-	0.23	0.07
80	10	R	0.25	0.22	0.84	0.36	0.11	0.48	0.23	0.01
120	10	R	0.25	0.22	0.86	0.26	-	-	0.28	0.07
160	10	R	0.25	0.22	0.84	0.23	-	-	0.38	0.05
240	10	R	0.25	0.21	0.84	0.31	-	-	0.43	0.06

5. CONCLUSIONS

Prototype regular road and railway bridges were designed to EC2 alone, or to EC2 and EC8, and their seismic vulnerability has been quantified. Bridges with deck supported on elastomeric bearings or monolithically connected to the piers, and hollow rectangular, wall-type or circular piers are covered. Compared to the design for gravity loads, seismic design gives much more vertical and transverse reinforcement in the piers and consequently higher flexural and shear strength. Piers designed for seismic resistance also have higher deformation capacity, but the increase is not proportional to the amount of reinforcement. The fragility of bridges designed to EC8 is satisfactory. Bridges designed to EC2 alone show acceptably low fragility, although markedly higher compared to EC8 designs. Hollow piers are vulnerable in shear; when the bridge is designed for gravity only, they may fail even before they yield. EC8 achieves uniform vulnerability for the different types of pier cross-sections examined in this work. Longer or taller bridges are in general less vulnerable. However, when the deck of railway bridges is monolithically connected to the piers, longer and taller bridges are more likely to exceed the deck horizontal deformation limits posed by (CEN, 2005c) for operation of the bridge. Bearings reach their ultimate damage state well before the piers, even before the piers yield.

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