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INELASTIC ANALYSIS OF RC BRIDGES AND APPLICATIONS TO RECENT EARTHQUAKES

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ABSTRACT

Seismic assessment of RC bridges requires the accurate calculation of both supply and demand for shear, flexure and axial deformation capacities. Since the use of full-scale testing is rather restrictive, advanced analysis assumes a role of extreme importance. In this paper, issues of assessment of supply are addressed, in terms of commenting on model adequacy and input motion characteristics. It is concluded that the inclusion of the foundation system in the model is very important, whilst the accurate evaluation of the soil stiffness characteristics is less important. Methods of selection and scaling of input motion are also discussed and comments on the use of natural earthquakes and their scaling to preserve a consistent energy input are offered. The significance of including the vertical component of earthquake ground motion in analysis is also discussed, in the context of axial and shear supply/demand assessment in the time domain. The models and methods presented are applied to two case studies from Kobe (Hanshin Expressway piers P663, P664 and P665 and the Fukae Honcho structure). It is indicated that the application of sound modelling assumptions and use of appropriate input motion components, coupled with representative response models, provides a powerful tool for unravelling failure modes of RC bridge structures.

KEYWORDS

Reinforced concrete bridges; ductility demand and supply; inelastic dynamic analysis; shear failure.

INTRODUCTION

Recent earthquakes, such as San Fernando (1971), Loma Prieta (1989), Northridge (1994) and Hyogo-ken Nanbu (1995) have emphasised the vulnerability of RC bridge structures to damage in strong earthquakes and the devastating effect this may have on the economy of the area affected. In addition to the hazard to human lives, the reduction in road transportation and elongated travel times impose a heavy toll on business, an effect that spreads well beyond the immediate vicinity of the earthquake-hit area. Moreover, the heavy damage inflicted to these important structures, which are usually subject to more stringent design and construction procedures than ordinary dwellings, undermines the public confidence in seismic design criteria. This is particularly true in Japan, in the wake of the recent earthquake in Kobe, where a very large number of structures either collapsed or suffered very heavy damage in an area considered to be highly active and where seismic design practice is amongst the most advanced world-wide.

To advance seismic design issues, accurate assessment of strength and ductility supply, as well as the maximum demand likely to be imposed, is essential. In this respect, assessment of supply quantities would require testing of individual piers under complex loading conditions representative of worst-case, yet realistic, flexure, shear and axial load components. Such tests, several of which have been undertaken at the University of California at San Diego (e.g. Seible *et al.*, 1994) as well as other places, are capital- and labour-intensive. It is clear, though, that parametric investigations could not be undertaken by experimentation and that including the foundation system is impractical. Furthermore, demand assessment dictates investigating bridge systems under realistic earthquake input. Such tests are currently underway at ISMES, (Italy) where three-pier scaled models are subjected to earthquake input through three independently-controlled shaking tables (Casirati *et al.*, 1995). Initial results from the tests have highlighted the extreme difficulty of satisfying dynamic similitude conditions with model materials and control of the three tables to the same level of precision, amongst other testing problems. It is therefore concluded also that experimental investigations are even more limited in scope in demand assessment than they are in supply studies.

The discussion of the role of testing in assessment of strength and ductility supply and demand of bridge structures leads to the conclusion that analysis has a most significant role to play, especially when considering the advancements of material constitutive modelling, structural analysis software and hardware platforms. Below, analytical aspects of pier-piles-soil modelling are briefly discussed, followed by a discussion of input motion selection and scaling. Two applications are presented from two most devastating earthquakes (Northridge, 17 January 1994 and Hyogo-ken Nanbu, 17 January 1995), where analytical investigations shed light on the likely reasons for the observed damage.

ANALYTICAL MODELS FOR ASSESSMENT OF DUCTILITY SUPPLY

Model Characterisation

Decisions regarding the detail required in model analysis of individual piers affect very significantly not only the obtained results but also computing effort. The simplest model of an RC bridge pier is a cantilever structure with a fixed base.

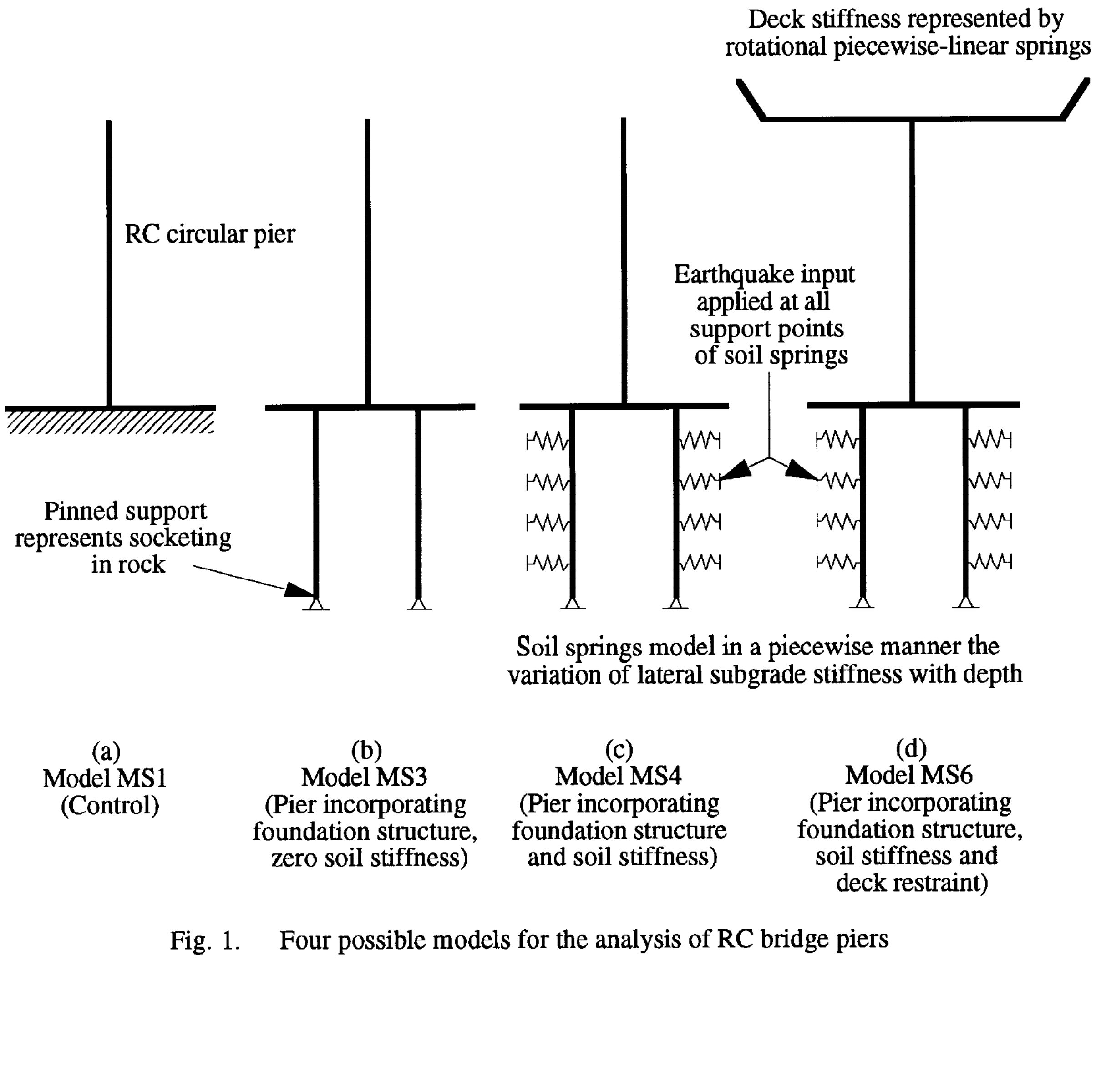

Fig. 1. Four possible models for the analysis of RC bridge piers

A further complication is including the piles-soil system, using a simple spring representation (this is not adequate for soil-related studies, but is used frequently in structural studies). In which case, decisions have to be made regarding the level of detail required in assessing soil characteristics to be modelled in the analytical investigation. Another modelling detail is whether and how the deck is connected to the pier. The existence of such a connection and its stiffness, has a very significant effect on the results obtained, since the deflected shape of a fully-restrained member is clearly distinct from that of a free standing cantilever. The former will have two potential plastic hinges, in contrast to the latter where only one zone will normally experience inelastic response.

Four possible models are shown in Figs. 1(a) through 1(d) above. The seismic response parameters of the ensuing models showed a high degree of variation (Elnashai and McClure, 1995). Table 1 below shows a sample of the large amount of results obtained. In the table, classes I, II and III pertain to various definitions of yield and ultimate limit states not discussed herein for brevity. Model MS5 has a deck restraint but no foundation modelling. Values for sub-models a, b and c are for three different levels of spring stiffness, given by the estimated value, half and twice the value, respectively.

Table 1. Response parameters for models presented in Fig. 1

	Displacement Ductility			Rotation Ductility			Curvature Ductility		
Model	Class I	Class II	Class III	Class I	Class II	Class III	Class I	Class II	Class III
MS1	4.2	4.5	6.3	8.0	8.8	13.4	8.1	9.0	13.8
MS3	3.0	3.1	3.9	8.0	8.8	13.1	8.2	9.0	13.5
MS4a	3.0	3.2	4.2	8.0	8.8	13.0	8.2	9.0	13.4
MS4b	3.2	3.4	4.4	8.0	8.8	13.3	8.2	9.0	13.6
MS4c	3.4	3.7	4.7	8.0	8.9	13.4	8.1	9.1	13.7
MS5	6.0	5.4	17.3	8.0	7.1	22.5	8.8	7.7	29.6
MS6a	2.5	2.5	4.0	8.0	7.1	26.9	8.7	7.8	37.9
MS6b	3.1	3.0	5.3	8.0	7.1	28.4	8.8	7.8	40.6
MS6c	3.4	3.3	5.7	8.0	7.2	28.3	8.8	7.7	41.0

The above sample of results not only indicates the differences to be expected in ductility supply assessment, but also point out, by comparison of pairs of rotation and curvature ductility, that critical sections are altered both in location and extent. It is also noticeable that, whereas the difference between the model with deck stiffness and no soil representation (MS5) and that where the piles and soil are represented (MS6) is more than 100%, doubling or halving the deck stiffness has a much less dramatic effect.

Input Motion Characteristics

Most seismic design codes require that more than one earthquake be used in analysis and that the set of records should be consistent with the code elastic response spectrum. The choice of whether to use natural or artificial records is a rather perplexing one, since on the one hand artificial records are almost invariably very rich in a wide frequency band whilst some natural records would be either over-conservative or unconservative depending on their predominant frequency content. To contribute to the debate on this subject, model MS6 was selected as the most detailed option for further inelastic dynamic analysis.

A set of six records selected to cover the full range of ratio of peak acceleration-to-velocity (Zhu *et al.*, 1988) were used. These were scaled using two approaches. The first was to scale to a common peak ground acceleration whilst the second utilises the area under the velocity spectrum of both the natural record and the code elastic spectrum for scaling. Several response parameters pertinent to seismic design were recorded (Elnashai and McClure, 1995), as given in Table 2 below, where values of ductility less than unity indicate elastic response.

Several interesting observations ensue from Table 2, such as the lower coefficient of variation of the set representing spectral intensity scaling (e.g. a 42% reduction in deflection ductility). It is also noticeable that the results vary significantly from different earthquakes.

Table 2. Response parameters from dynamic analysis with two scaling methods

Earthquake Record	Direct pga Scaling Displacement Ductility Demand	Direct pga Scaling Curvature Ductility Demand	Spectral Intensity Scaling Displacement Ductility Demand	Spectral Intensity Scaling Curvature Ductility Demand
Friuli	0.27	0.40	0.28	0.50
Gazli	0.75	0.80	1.55	2.00
Loma Prieta EW	2.20	4.20	3.20	8.00
El Centro	1.60	2.40	2.20	3.80
Spitak	1.60	2.50	2.60	5.50
Loma Prieta NS	2.75	6.50	2.70	6.00
Mean	1.78	3.28	2.45	5.06
Std. Deviation	0.67	1.94	0.55	2.03
Coeff. of Variation	0.38	0.59	0.22	0.40

Of particular interest are Friuli and El Centro; the former is one of the popular European records whilst the latter has been used very widely by many researchers as the sole assessment tool. The results for Friuli are attributable to its narrow band frequency content which is also far removed from the period of the structure. The El Centro results are more a function of the time-history, where there is a single pulse of large amplitude and a specific frequency. This emphasises the necessity of careful selection of input motion lest unconservative results are obtained.

SHEAR AND AXIAL FORCE VARIATION

Effect of Axial Force on Shear Supply and Demand

Due to the brittle nature of shear failure, seismic design of RC structures employs capacity design procedures to estimate the shear demand at the attainment of the maximum flexural strength, including sources of overstrength. It is, however, rare that assessment of shear demand and supply takes into account the axial force variation during strong shaking, due to both horizontal and vertical ground motion components. The best available simple model for shear assessment is that by Priestley *et al.* (1994):

$$V_{tot} = V_c + V_s + V_p \tag{1}$$

where

$$V_c = 0.8A_{gross}k\sqrt{f'_c} \tag{2}$$

$$V_s = \frac{\pi}{2}A_{sp}\frac{f_{ys}\,D'}{s}\cot 30 \tag{3}$$

$$V_p = \frac{P\,D}{L} \tag{4}$$

In these equations, A_{gross} is the cross-sectional area of the concrete, k reflects the degradation of shear strength with increasing displacement ductility, D and D' are the diameters of the pier and the confined concrete core, L is the height of the pier and P is the axial load acting on the pier. The following equations describe the variation of k with pier displacement ductility, μ_Δ:

$$0 \le \mu_\Delta \le 1 : k = 0.29 \tag{5}$$

$$1 \le \mu_\Delta \le 3 : k = 0.29 - 0.095(\mu_\Delta - 1) \tag{6}$$

$$3 \le \mu_\Delta : k = 0.10 \tag{7}$$

The model is based on mechanical analogues of the shear resistance systems, is simple and is hence attractive for assessment purposes. By virtue of the inclusion of the effect of axial force and displacement ductility, it is amenable to implementation in the time domain. This has been undertaken at Imperial College for bridges (Dodd *et al.*, 1994) and buildings (Georgantzis, 1995). In the course of analysis of a large curved RC bridge, the former publication reports the variation in demand and supply as a function of time, with and without the inclusion of the vertical earthquake component, as shown in Fig. 2 below.

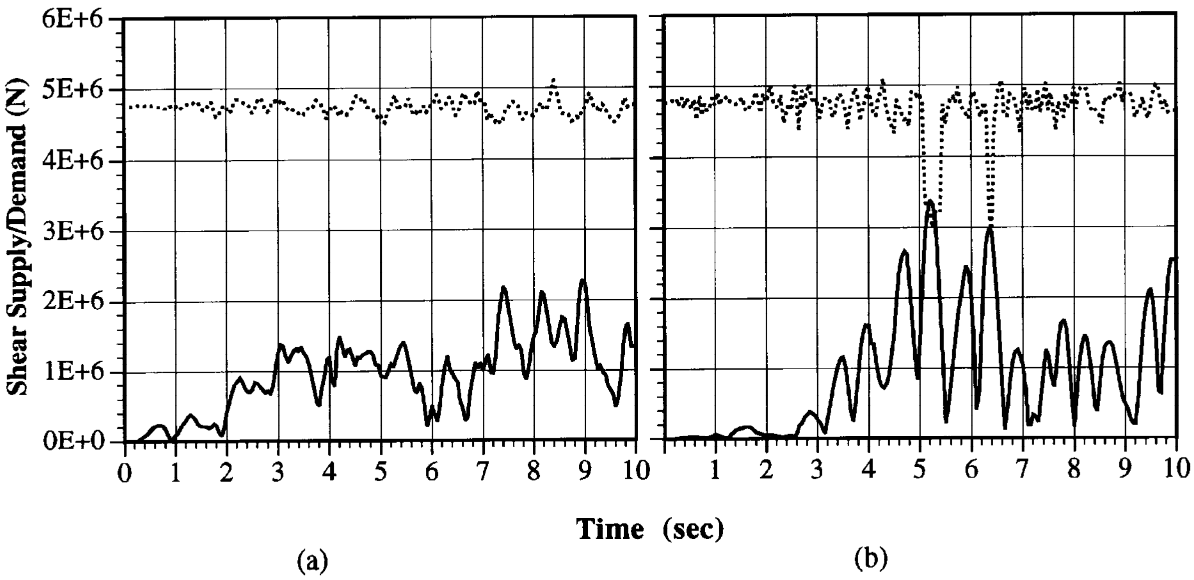

Fig. 2. Shear supply (dotted) and demand (solid) under (a) transverse and (b) combined transverse and vertical earthquake motion for a large curved RC bridge (adapted from Dodd *et al.*, 1994)

The above time-history results indicate that both supply and demand vary with time and are seriously affected by the vertical earthquake motion. It is therefore evident that shear safety assessed on the basis of an average axial force and ductility demand may be unconservative. The same applies to shear assessment in the absence of vertical earthquake motion.

Effect of Vertical Motion on Compressive and Tensile Forces

The axial force in piers due to dead and live loads is included in the static design, whilst no consideration is given to dynamic earthquake forces. This is usually justified on the basis of the high safety factor against compressive failure. Such an approach ignores the dynamic nature of earthquake forces, which is applied upwards as well as downwards, and under-estimates the additional dynamic compression force that may be exerted on piers subjected to high vertical vibrations. These effects were studied by Elnashai and Papazoglou (1995) using a data set of natural records with high vertical component (pga ≥ 0.30g). An example of vertical spectra obtained is shown in Fig. 3 below.

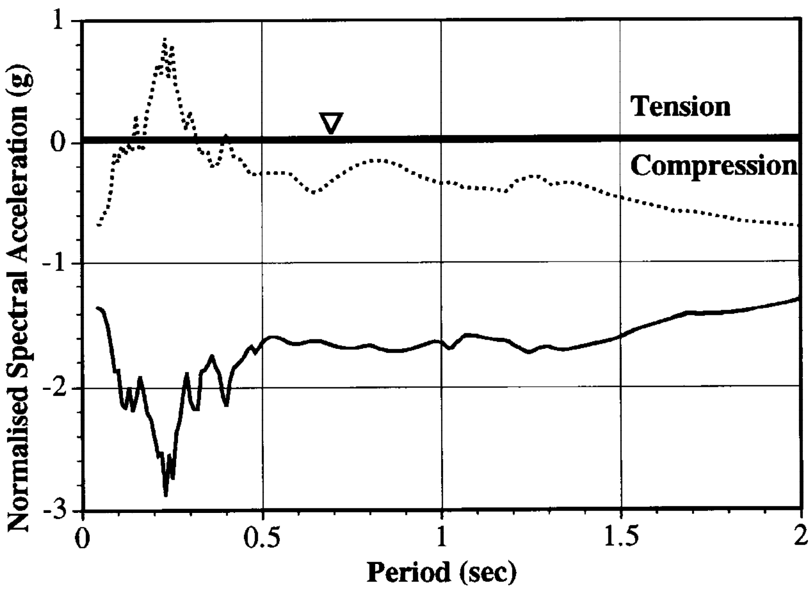

Fig. 3. Vertical spectrum for JMA Kobe (17 January 1995), minimum (solid) and maximum (dotted) response using bilinear stiffness; static load (1g) included (from Elnashai and Papazoglou, 1995)

Figure 3 demonstrates that high vertical compressive forces may be exerted on structures, up to 3g in total. This may cause reduction in flexural ductility as well as potential compressive failure.

CASE STUDIES OF ANALYTICAL BRIDGE ASSESSMENT

Hanshin Expressway Fukae-Honcho Structure (piers 124-142)

This structure attracted considerable attention following the Hyogo-ken Nanbu earthquake of 17 January 1995, due to its spectacular failure mode and the effect its closure has had on transportation in the busy Higashi Nada Ward area. This is a circular RC single column pier structure supporting four traffic lanes. The pier section is 3.1m in diameter and about 15m in length, whilst the deck has a width of 20.5m. The structure suffered total collapse in about 16 piers. Rupture seemed to have occurred at sections at about 2.5m above base level, where the two layers of reinforcement were reduced to one. Modelling of the structure and details of material properties used are given elsewhere (Elnashai *et al.*, 1995). A reduced structure comprising three piers and two deck spans was subjected to the three components of the JMA Kobe record. The yield and ultimate displacements of the piers were about 60mm and 310mm, respectively, leading to a displacement ductility supply of about 5.0. Inelastic dynamic analysis results gave an imposed demand of 5.0-5.3. Comparison with the results from a rapid assessment by Seible *et al.* (1995) is given in Table 3 below alongside material strength values used in each analysis.

Table 3. Comparison between analytical results of Seible *et al.* (1995) and Elnashai *et al.* (1995)

	Concrete Strength (MPa)	Steel Strength (MPa)	Yield Displ. (mm)	Critical Concrete Strain	Ultimate Displ. (mm)	Displ. Ductility	Ultimate Moment (kNm$\times10^4$)
Elnashai *et al.*	26.5	300	62	0.007	309	4.98	6.48
Seible *et al.*	35.0	350	57	0.005	134	2.36	6.09
Difference	25%	16%	8%	40%	130%	110%	7%

The study by Seible *et al.* impressively provided rapid assessment of the possible causes of failure by making use of equivalent static concepts and analytical expressions for plastic hinge length, effect of yield penetration and tension shift on member length and effective length, respectively, and critical strain values. The differences observed in Table 3 are attributed to:

i. Differences in strength parameters.
ii. Differences in the definition of the critical strain value.
iii. Seible *et al.* make assumptions on plastic hinge length and yield penetration, whilst the analysis of Elnashai *et al.* automatically obtains the former effect from the detailed model of the pier.
iv. The ultimate displacement derived by Seible *et al.* assumes a rectangular distribution of curvature in the plastic hinge zone. Such an assumption is not needed in the other study where inelasticity is allowed to spread along the member length and across its depth.

The above list is not exhaustive. Considering the fundamental differences of approach in the two studies, the results obtained are surprisingly close, especially for strength values and yield limit state. The comparison also highlights the importance of refined analysis, whilst not down-grading the significance of rapid assessment.

With regard to the behaviour of the structure, Seible *et al.* conclude that the shear capacity of 6.4×10^3 kN will be reached prior to the attainment of the flexural capacity of 5.4×10^3 kNm, hence shear failure will occur. It was also stipulated that a response acceleration of 1.20g is needed to cause the exceedence of the displacement ductility capacity, which is attainable when considering that the peak ground acceleration for the JMA Kobe record exceeds 0.85g. The study undertaken at Imperial College indicated that the displacement ductility demand evaluated from a 3D analysis of a two pier model equals or exceeds the supply calculated from detailed inelastic analysis, as given above. Both studies agree that the piers possessed inadequate strength and deformation capacities to resist the demand imposed by the high levels of ground shaking.

Piers P663, P664 and P665 of the Hanshin Expressway Public Corporation Structure

The exact location of this structure is not known. Details of the sections, structural system and reinforcement are given elsewhere (Elnashai *et al.*, 1995). Herein, the objective is to demonstrate the effective role played by advanced analysis in seismic assessment. The structure comprises single column circular RC piers of diameter 2.3-2.5m supporting six steel girders which in turn support the deck slab. The structure suffered a rather peculiar mode of failure (reason for its selection by the Imperial College field mission group), whereby a symmetric ring of crushed concrete was observed at mid-height, with buckling of reinforcement. The three piers studied suffered varying degrees of distress, with piers P663 and P664 damaged much more significantly than pier P665. The values of static and dynamic axial force, their variation in time and their effect on shear and flexural capacities were deemed of sufficient significance, as shown in Fig. 4, to warrant special consideration.

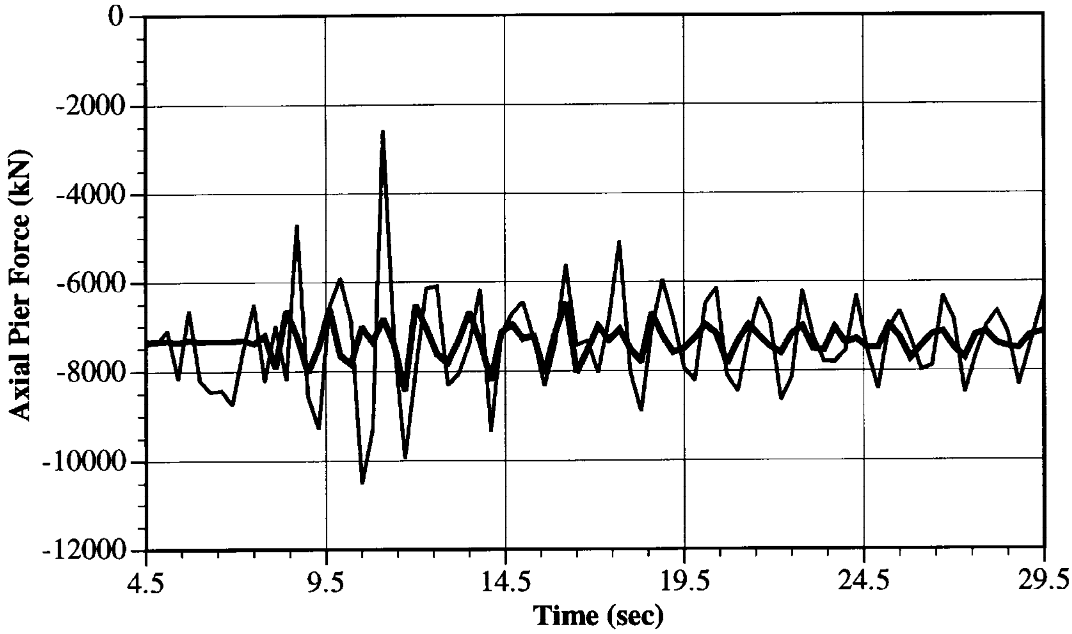

Fig. 4. Axial force variation in pier P663 excluding (thick line) and including (thin line) the effect of the vertical component of earthquake motion

The strength characteristics of the piers, evaluated from static push-over analysis taking into account the extreme values of static and dynamic axial force, are given in Table 4 below. All strength properties are evaluated at the bottom section and at the location of the reinforcement cut-off.

Table 4. Strength characteristics of piers including effect of dynamic axial force

	Max. Ultimate Moment ($kNm\times10^4$)		Min. Ultimate Moment ($kNm\times10^4$)		Ultimate Shear Force ($kN\times10^3$)		Ultimate Axial Force ($kN\times10^3$)	
Pier	Bottom	Middle	Bottom	Middle	Bottom	Middle	Bottom	Middle
P663	6.3	2.8	6.1	2.6	5.8	5.3	174.0	170.0
P664	7.4	3.2	6.9	3.0	7.1	6.4	198.0	194.0
P665	7.2	3.1	6.9	3.0	6.7	6.0	198.0	194.0

Assessment of the earthquake analysis results, applying the JMA Kobe record, showed that piers P663 and P664 are close to shear failure, but the observed failure mode is not shear. Moreover, the results also indicated that piers P664 and P665 are more stressed in flexure than P663, yet the observed damage shows that pier P665 suffered very much less distress than P663 and P664. To attempt to explain this, the axial force and capacity were examined, by plots similar to Fig. 3. The applied axial force is a fraction of the axial capacity for all piers. However, piers P663 and P664 are subjected to significantly higher compressive forces than P665 at the middle section (6.1×10^3 kN for P663, 8.6×10^3 kN for P664 and 5.1×10^3 kN for P665). These forces also fluctuate more for the former two piers than in the case of the latter throughout the time-history of response. It was therefore postulated in the Imperial College study that the observed damage pattern is due to the combined effect of all three components: shear, flexure and axial actions. In this respect, the higher axial force may have caused cover spalling, hence reinforcement buckling. Minor site effects may have played a significant role, with regard to long-term relative settlement which will tend to alleviate the load off the settled pier, thereby increasing it on adjacent piers. This analysis strengthens the case for refined inelastic dynamic analysis using all three components of earthquake motion in seismic assessment of RC bridges, since the axial force is shown to play a significant role.

CLOSURE

The studies summarised above, undertaken at Imperial College, indicate that seismic analysis of bridge piers is significantly affected by the boundary conditions and model details, but is less sensitive to the actual value of stiffness at the boundary (e.g. soil spring stiffness or pier-deck connection stiffness). Moreover, the ductility demand imposed is very highly affected by the characteristics of the input motion. A carefully selected set of natural records, with a minimum of two in each of the ranges of a/v (where a and v are the peak ground acceleration and velocity, respectively) low, medium and high, provide a reasonable basis for analysis. Also, scaling the records using velocity spectral intensity is far superior to the use of direct acceleration scaling. It has also been demonstrated that assessment of shear capacity and demand, accounting instantaneously for the axial force variation and the flexural ductility imposed, is a very powerful tool. In this respect, use of the vertical earthquake component of motion is essential.

The case studies presented, both from the recent earthquake in Kobe, highlight the very important role played by advanced analysis, to furnish detailed information on supply and demand. When accurate models are used, alongside effective solution techniques, under realistic three-component earthquake motion, the results correlate very well with observed damage. The analysis also sheds light on possible causes of damage that would have been otherwise very difficult to interpret.

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