INFLUENCE OF FOUNDATION COMPLIANCE ON THE SEISMIC RESPONSE OF BRIDGE PIERS

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SNYOPSISIS

The paper summarises the results of dynamic analyses of some single-stem and double-stem bridge piers responding to a range of earthquake ground motions, including both natural and synthetic earthquake records. The influence of foundation flexibility was modelled by an 'extended leg' analogy and by a more refined approach in which the soil is replaced by an equivalent spring system. A bilinear moment-curvature loop was assumed to model the plastic hinge behaviour. The curvature ductility factor demand of bridge piers with different foundation flexibilities was determined and compared with the rigid foundation case.

INTRODUCTION

Satisfactory performance of major highway and rail bridges under severe seismic attack is clearly of prime importance. As well as the obvious requirements of minimising loss of life and protecting capital investment, aims which are held in common with buildings, a further requirement, that of providing access to the affected area during post-earthquake rescue and rehabilitation operations, will commonly exist. If a road bridge is to be part of a critical transportation lifeline it will need to be functional, at least to emergency (4-wheel drive) vehicles, within hours of the earthquake. Damage to rail bridges during design-level earthquakes should not involve more lengthy repair work than necessary for ground-supported track.

These considerations effectively imply better performance of major bridge structures under seismic attack than would be required for Class I buildings⁽¹⁾. This philosophy is inherent in the current Ministry of Works and Development (MWD) specifications for highway bridge design⁽²⁾ which set higher seismic design base shear coefficients and design levels of ductility than specified for buildings⁽¹⁾.

Generally it is uneconomic to design bridge structures to respond in the elastic range to design level earthquakes, and a ductile design is necessary. Inelastic response of bridge structures under seismic attack will generally involve plastic hinging of the bridge columns, unless mechanical energy dissipators are incorporated in the design. In most cases the superstructure design is governed by dead and live load considerations and it is both impractical and undesirable to design for plastic hinging in superstructure members. Behaviour is consequently different in concept from that required of building frames, where a

capacity design approach is adopted to ensure beam hinging by specifying column flexural and shear strengths to be higher than the maximum input associated with beam hinges forming at maximum feasible beam strength.

By their nature bridge piers tend to be simple structures involving few structural members. A typical New Zealand bridge pier consists of one or two columns, or a wall, supported by either a footing or a pile cap connected to driven or bored piles. At the top of the pier is a substantial pier cap or hammerhead which supports the deck structure. The chosen energy dissipating mechanism involves plastic hinges in the piers rather than the foundation (pile cap and piles) because of the uncertainty of hinge location in piles and the greater accessibility for inspection and repair of the piers. For wall piers or single-stem piers under seismic loading acting at right angles to the longitudinal axis of the bridge, plastic hinge regions will normally be only at the pier base. However, with multiple-stem piers, or single-stem piers monolithic with the superstructure and loading in the direction of the longitudinal axis of the bridge, plastic hinging may develop top and bottom in the piers.

Satisfactory design depends on an understanding of the ductility demands of the piers when responding inelastically to severe earthquake loading. This paper examines the significance of foundation flexibility and earthquake characteristics to the ductility demands of bridge piers.

THEORETICAL CONSIDERATIONS OF FOUNDATION COMPLIANCE

Foundation compliance in the form of translation or rotation of the footing or pile cap beneath the pier will have the effect of increasing yield displacement at the centre of mass of the bridge superstructure above the value corresponding to flexural deformation alone. Other displacement contributions may be made by shear deformation of bearings where the superstructure is supported at the pier through elastomeric bearing pads, and shear deformation of the pier itself.

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An obvious effect of the increased flexibility from the above causes will be to lengthen the fundamental period of vibration above that corresponding to flexural deformation of the pier alone. A less obvious effect is a radical change in the relationship between the overall displacement ductility factor and local curvature ductility factor at the plastic hinges. Consider the system shown in Fig. 1 representing simple single-stem bridge piers. The argument developed below is equally applicable to multiple-stem piers with plastic hinges forming at the top and bottom. The pier in Fig. 1a has a rigid foundation and monolithic pier/superstructure construction. Assuming for simplicity an elasto-plastic yield characteristic, the plastic rotation, considered to be concentrated at the centre of the plastic hinge at the base of the pier, will be

$$\theta_{p1} = \frac{(\mu - 1)\Delta_s}{L_p - \frac{L_p}{2}} \quad (1)$$

where $\Delta_s = \Delta_y$ is the displacement of the centre of mass at first yield resulting from flexural deformation of the pier, L_p is the plastic hinge length, and μ is the displacement ductility factor.

The average plastic curvature over the hinge length will be

$$\phi_{p1} = \frac{(\mu - 1)\Delta_s}{L_p(L - \frac{L_p}{2})} \quad (2)$$

Now assuming a triangular bending moment diagram in the pier due to horizontal seismic load concentrated at the centre of mass of the pier superstructure, the displacement due to pier flexure at first yield is

$$\Delta_s = \frac{M_y L^2}{3EI}$$

where M_y is the yield moment. Therefore the yield curvature is

$$\phi_y = \frac{M_y}{EI} = \frac{3\Delta_s}{L^2} \quad (3)$$

The curvature ductility factor is defined as

$$\begin{aligned} \left[\frac{\phi_u}{\phi_y} \right]_1 &= \frac{\phi_y + \phi_{p1}}{\phi_y} \\ &= 1 + \frac{(\mu - 1)\Delta_s}{L_p(L - \frac{L_p}{2})} \cdot \frac{L^2}{3\Delta_s} \\ &= 1 + \frac{(\mu - 1)}{1.5 \frac{L_p}{L} (2 - \frac{L_p}{L})} \end{aligned} \quad (4)$$

The relationship between ϕ_u/ϕ_y and μ given by Eq. 4 is similar to that given for the equivalent case in the MWD's publication on pier ductility (3).

The pier in Fig. 1b is founded on a

single cylinder pile, and the superstructure transmits vertical and shear loads to the hammerhead and pier through elastomeric bearing pads. The yield displacement is thus increased by translation (Δ_T) and rotation (Δ_r) of the pile cap, and by shear deformation of the bearing (Δ_b). Thus

$$\Delta_y = \Delta_T + \Delta_r + \Delta_b + \Delta_s = C \cdot \Delta_s \quad (5)$$

where C is a coefficient representing the increase in the elastic flexibility of the system. A capacity design procedure is used to ensure formation of the plastic hinge at the pier base rather than in the foundation cylinder. Assuming a bilinear moment-curvature relationship for the hinge as before, all additional deformation after yield must result from plastic rotation of the pier hinge. If the displacement ductility factor μ is the same for the rigid base case and the flexible base case, then by analogy to Eq. 1 it is necessary that the plastic rotation in the pier at the base of the flexible case should be

$$\theta_{p2} = \frac{(\mu - 1)C\Delta_s}{L_p - \frac{L_p}{2}} = C \cdot \theta_{p1} \quad (6)$$

and therefore that $\phi_{p2} = C \cdot \phi_{p1}$

The yield curvature at the plastic hinge ϕ_y is still governed by Eq. 3 and therefore remains unaffected by the increased flexibility of the total system. Hence the curvature ductility factor for the flexible case is

$$\begin{aligned} \left[\frac{\phi_u}{\phi_y} \right]_2 &= \frac{\phi_y + \phi_{p2}}{\phi_y} \\ &= 1 + \frac{C(\mu - 1)\Delta_s}{L_p(L - \frac{L_p}{2})} \cdot \frac{L^2}{3\Delta_s} \\ &= 1 + \frac{C(\mu - 1)}{1.5 \frac{L_p}{L} (2 - \frac{L_p}{L})} \end{aligned} \quad (7)$$

Hence, if the displacement ductility factor μ is the same for the rigid base case and the flexible base case, then

$$\left[\frac{\phi_u}{\phi_y} \right]_2 = 1 + C \left[\left[\frac{\phi_u}{\phi_y} \right]_1 - 1 \right] \quad (8)$$

Since for stiff piers the value of C may be as high as 5 or more, the effect on curvature ductility demand may be substantial. For the above argument to be valid, it is of course necessary that the two systems be designed for the same value of μ . This will generally imply a lower design base shear coefficient for the more flexible pier of Fig. 1b as a result of the increase in elastic period, which will be governed by the relationship

$$T_2 = \sqrt{C} \cdot T_1 \quad (9)$$

COMPUTER ANALYSES OF BRIDGE PIER MODELS

Pier Characteristics

The theoretical approach outlined in the previous section is a fundamental part of current MWD practice for the seismic design of bridge piers⁽³⁾. That is, the displacement ductility factor demand μ is assumed to remain constant and hence the curvature ductility factor demand for a flexible system may be very high. In order to check the validity of the arguments involved in developing Eq. 8 an investigation was made⁽⁴⁾ of the interaction between additional flexibility earthquake characteristics, and ductility demand for four bridge pier models using a simple representation of the effects of foundation flexibility. Additional elastic flexibility resulting from pier shear deflection or bearing deformation were not included in the study, but it is reasonable to assume that trends indicated from foundation analyses would also apply to the other cases.

The four pier models investigated included one double-stem pier (type A) and three single-stem piers (types B,C,D). In each case an octagonal pier stem cross section of 1.5m diameter was used. Concrete compressive cylinder strength and modulus of elasticity were taken as $f'_c = 27.6$ MPa and $E_c = 24.9$ GPa, and the reinforcing steel was assumed to have a yield strength of $f_y = 276$ MPa. The minimum concrete cover to the longitudinal reinforcement was 50 mm. The stem was assumed to have a moment of inertia of $I = 0.108$ m, which is 43% of the uncracked value.

The computer models used for the analysis of rigid-base conditions are shown in Fig. 2. In each case a rigid end block of 2.04 m was specified between the pier and the centre of mass to represent the stiff pier capping beam and diaphragm. The height of the centre of mass above the foundation was chosen to give natural periods of approximately 0.2, 0.5, 1.0 and 2.0 seconds for pier types A to D, respectively. A lumped weight of 2,667 kN at the top of the pier was assumed for the vertical degree of freedom, this being a realistic value for a 20 m span of typical bridge deck. For the horizontal degree of freedom a lumped weight of 4,000 kN was assumed at the top of the pier, representing one and one half spans (end span plus half interior span), together with a distributed weight of 45 kN/m up the height of the pier. Earthquake loading was taken as acting at right angles to the axis of the bridge.

The required yield moment (plastic hinge) capacities were calculated in accordance with the MWD Highway Bridge Design Brief⁽²⁾ requirements for Zone A, which specifies an ultimate base shear force of

$$V_u = 1.35 \text{ CFW} \quad (10)$$

where 1.35 is the load factor, C is the basic seismic coefficient⁽²⁾ for Zone A, F is the importance factor (taken as 1.0) and W is the load associated with the horizontal degree of freedom.

Dynamic inelastic time-history analyses were carried out using a program TWODEE

developed by Sharpe⁽⁵⁾, with a time-step of 0.005 seconds and 5% viscous damping associated with the first two modes of vibration. The moment-curvature relationship was represented by a bilinear function with a bilinear factor of either 0.032 or 0.0384. Plastic hinge lengths were based on Baker's formula⁽⁶⁾. Table 1 summarises the properties of the piers that were not constant for all piers.

'Extended Leg' Simulation of Foundation Flexibility

Rather than attempting to accurately model exact soil behaviour, the investigation concentrated on the general consequences of foundation flexibility. The simple computer model adopted to take into account foundation flexibility involved the concept of an 'extended leg' beneath the pier, as shown in Fig. 3. For each 'extended leg' member the stiffness was the same as that of the pier member but the strength was sufficiently high to ensure plastic hinge formation at the base of the pier rather than within the 'extended leg'. The boundary conditions specified for the pier base node determine whether the foundation flexibility is due to translation, rotation, or a combination of translation and rotation of the pier base node. For the portal frame model (type A in Fig. 3) only translation of the pier base node was allowed (not rotation), but for the single-stem pier models (types B, C and D in Fig. 3) the pier base node was allowed to rotate and translate.

Increasing the length of the 'extended leg' beneath the pier increased the flexibility coefficient C defined in Eq. 5. For the case of pier types B to D, the natural period will be given by

$$T = \sqrt{\frac{(W/g)}{3EI/(L + L_{ex})^3}} \quad (11)$$

where L_{ex} is the length of the extended leg, assuming a linear moment distribution down the pier. Thus

$$T^2 \propto (L + L_{ex})^3 \quad (12)$$

From Eqs. 9 and 12 the relationship between the flexible foundation and rigid base conditions may be expressed as

$$\left(\frac{T_2}{T_1}\right)^2 = C = \frac{(L + L_{ex})^3}{L^3}$$

giving the length of the extended leg as

$$L_{ex} = L (\sqrt[3]{C} - 1) \quad (13)$$

In this investigation values for C of 1, 2, 3 and 4 were considered. Note that C = 1 corresponds to the rigid foundation case. In each case the yield moment of the base hinge was kept at the value appropriate to the rigid base natural period. That is, no allowance was made for change in the base shear coefficient with increase in natural period resulting from foundation flexibility. This enabled the influence

of foundation flexibility on particular bridge piers subjected to specified earthquake attack to be investigated, corresponding to the previously described theoretical approach. It was felt that this represented the most fundamental approach.

Earthquake Records

The five earthquake records used for dynamic analyses consisted of:

El Centro N-S 1940	first 10 seconds.
Taft N69W 1952	first 10 seconds.
Bucharest N-S 1977	first 10 seconds.
Artificial B1	first 20 seconds.
Artificial A1	first 30 seconds.

The artificial earthquakes are from the suite proposed by Jennings, et al⁽⁷⁾. Not all earthquake records were run for all combinations of flexibility coefficient C and pier type.

Results of Computer Analyses

The resulting values of displacement and curvature ductility factor demand found from the dynamic analyses are listed in Table 2, which includes for comparison calculated requirements based on Eqs. 4 and 7 and the displacement ductility factor specified by the Highway Bridge Design Brief⁽²⁾ for the chosen values of base shear coefficient and natural period. The required structure displacement ductility factor is 6 for the rigid base cases, but has been reduced to less than 6 where appropriate in the flexible base cases in proportion to the strength excess provided as a result of the increase in period.

For pier type A the curvature ductility factor is shown to increase with increasing foundation flexibility for all four earthquake records run, at a faster rate than predicted by Eq. 7. This trend is probably due to the fact that the increase in period associated with foundation flexibility moves the type A pier into a region of higher acceleration response on the response spectra for the earthquakes, as indicated in Fig. 4 by the shift from point S1 to S2. For pier types B, C and D the curvature ductility factor demand is comparatively insensitive to increasing foundation flexibility. In many cases the curvature ductility factor demand shows a decrease rather than the increase predicted by Eq. 7. The exception is the response of pier type B to the Bucharest record, which has a maximum in the acceleration response spectra at about 1.0 - 1.5 seconds. It should be noted that the Bucharest record used for analysis was an early, coarse digitization with a Nyquist frequency of about 2.5 Hz, and the results for the stiff pier type A model may be significantly in error.

It appears that where the increase in foundation flexibility shifts the natural period into a region of reduced acceleration response, as indicated in Fig. 4 by the shift from point F1 to F2, the requirements of Eq. 7, which assumes an equal displacement ductility factor demand for rigid base and flexible base systems, are excessively severe, and analysis based on a constant plastic displacement (and hence constant curvature ductility factor) might be more reasonable.

In order to check whether the reduced response for flexible foundation conditions apparent in Table 2 for medium and long period piers was purely a function of the earthquake response spectra characteristics, a pier model similar to the pier type C was analysed under the El Centro 1940 N-S record, with a plastic hinge location at node 2 (Fig. 5B) corresponding to the theoretical position to simulate a foundation flexibility coefficient of $C = 2$. Results were compared with those from analysis of the identical pier, but with the plastic hinge location at node 1 (Fig. 5a). Thus Fig. 5a represents a rigid base condition, while Fig. 5b represents a shorter pier of the same section whose foundation flexibility results in identical elastic response to the longer rigid base pier. The moment capacities at the two plastic hinges were in proportion to the distance from the centre of mass, so that yielding initiated at 0.137 g response in each case. In Fig. 5a hinging was prevented at node 2 by specifying a high yield moment capacity for the upper portion of the pier. Similarly in Fig. 5b hinging in the 'extended leg' portion (node 1 $\rightarrow$ Node 2) was inhibited. The plastic hinge length was set at 0.488 m and a bilinear moment-curvature relationship with a second slope ratio $R = 0.032$ was adopted for both analyses. It should be noted that if identical displacement response at the centre of mass is to result, the plastic rotation θ_p (and hence the curvature ductility demand) at node 2 in Fig. 5b will be higher than at node 1 in Fig. 5a.

Displacement time-histories for the first 10 seconds of response are compared in Fig. 6. It will be observed that until yield occurs at 1.85 seconds, the displacement responses are identical, but that subsequently the response curves differ significantly with the peak response for the base-hinge case of 88 mm considerably exceeding the peak response of 70.5 mm for the case with the hinge at node 2. Displacement ductility factors for the two cases were 3.32 and 2.66 respectively, representing a 20% reduction in peak response and a 28.5% reduction in peak plastic displacement. In fact, the plastic curvature for the case in Fig. 5b rather than being larger than that for the case in Fig. 5a, as would be required for equal total displacement, was actually 1% lower.

It seems that the reduction in peak response may have been a result of increased hysteretic damping and slightly increased pier moments associated with the higher curvature ductility factor in the case of Fig. 5b (resulting from the reduced yield curvature rather than an increase in plastic curvature). This effect may have contributed to the low response levels noted in Table 2.

Realistic Simulation of Foundation Compliance

The 'extended leg' model used to simulate foundation compliance in the analyses described in the previous section can be expected to indicate general trends, but is not directly applicable to specific cases, nor can it adequately model the effects of soil yielding. In order to investigate these aspects further, a more realistic simulation of an actual bridge

was made, with direct modelling of the pile system, and simulation of the soil by equivalent inelastic springs.

The structure adopted for analysis was the Mountain Stream bridge in Westland, designed by the MWD, and shown in Fig. 7. Each reinforced concrete pier is of single-stem, founded on a single reinforced concrete cylindrical pile embedded in coarse sandy gravel. The actual bridge pier was modelled using a pier type B with the properties shown in Table 1 and given earlier in this section. This pier type B had properties which are very similar to the actual Mountain Stream bridge pier. Fig. 8 shows the configuration of the pier-pile system and the computer model. The soil up the 7.62 m height of pile is replaced by eleven spring members numbered 2,4,6,....,22. Each of these spring members replaces a 0.7 m thick layer of soil except the top spring member 22 which replaces a 0.62 m thick soil layer. The pile is divided into 12 members numbered 1,3,5,....,23. Member number 24 is the pier type B.

Simulation of the soil as a series of springs ignores the effects of soil shear deformation, and the assumption that nodes 1,3,5,7, etc. are boundaries implies that the free-field soil displacements under seismic attack are equal to all heights. Although these assumptions are questionable, it is felt that the approach adopted is currently the best approximation. The shear-beam model⁽⁸⁾, which enjoyed a vogue for some years, and which would impose differential ground displacements at the soil boundary nodes is no longer considered an adequate representation of soil behaviour⁽⁹⁾.

An assumed profile of the soil modulus E_{soil} is required to determine the stiffness of the spring members. For granular soils the best approximation appears to be a soil modulus which increases linearly with depth⁽¹⁰⁾. On this basis

$$E_{\text{soil}} = n_h z \quad (14)$$

where z = depth of soil and n_h = horizontal subgrade reaction coefficient = 10.7 MN/m^3 for dense submerged granular soil, according to Terzaghi⁽¹¹⁾. The lateral pile force was assumed to spread uniformly from the pile over a wedge shaped soil sector with sides at 45° to the direction of load and on this basis the stiffness of the spring member was shown to be⁽⁴⁾ approximately

$$K = n_h z t \quad (15)$$

where t = thickness of soil layer. The reduction in n_h due to cyclic loading and the increase in n_h due to increased relative density of soil surrounding the pile due to driving were not considered. These two effects may well compensate each other.

The lateral soil pressure at yield was assumed to be three times the passive soil pressure calculated using the Rankine method. This followed results reported by Prakash⁽¹²⁾ and Broms⁽¹³⁾. Accordingly, the lateral soil reaction per unit length of pile at yield is

$$Q_y = 3D\gamma K_p z \quad (16)$$

where D = pile diameter, γ = unit weight of soil, z = depth, and K_p is coefficient of passive Rankine earth pressure

$$K_p = \frac{1 + \sin\phi_d}{2 - \sin\phi_d} \quad (17)$$

where ϕ_d = angle of internal friction of the cohesionless soil, assumed to be 33° for the Mountain Stream bed material.

With a static horizontal load applied at the centre of the mass of the superstructure at the yield strength of the pier, the horizontal displacement of the load was found to be 93.9 mm. The yield displacement at the load for pier type B due to pier bending alone was 10.52 mm. Hence the foundation flexibility coefficient for this pier-pile system was $C = 93.9/10.52 = 8.9$. Thus the foundation of this bridge is very flexible. The top 3 m of soil was found to be exerting a lateral pressure which exceeded the available soil resistance.

Two dynamic analyses were carried out. In one analysis the springs (cantilevers) and pile were given artificially high yield strengths which ensured that the foundation responded elastically and hence that yielding could occur only in the pier member. In the other analysis the cantilever yield strengths were based on the soil yield strength from Eqs. 16 and 17 (and therefore the soil behaved elasto-plastically) but the pile was given an artificially high yield strength which ensured that the pile responded elastically, and hence that yielding could only occur in the pier member and the soil. Both analyses were conducted using the horizontal component of the El Centro N-S 1940 earthquake record. Viscous damping was assumed to be 5% of critical. Fig. 9 shows the time history of the lateral displacement of the centre of mass of the superstructure. Clearly the elasto-plastic soil model gives a lower response than the elastic soil model. The analysis of the soil elasto-plastic model showed that the top five spring members yielded, corresponding to yielding of soil for about 3.4 m from the surface.

The response of the soil yield model in Fig. 9 indicates that the soil is effectively providing additional damping. The extent of the additional equivalent viscous damping was investigated by specifying a higher percentage damping with the soil elastic model and, by a trial and error process, the percentage damping was found in which the soil elastic model gave the same maximum displacement as the soil elasto-plastic model with damping taken as 5% of critical. To save computer time, the soil elastic model used was in fact the extended leg model with the same coefficient of foundation flexibility as the more realistic spring member foundation flexibility model. It was found that soil yielding in this particular pier-pile system is equivalent to increasing the damping from 5% to 9%. However it should be noted that the elasto-plastic soil model overestimates the area within the hysteresis loop of the actual soil, since the soil/pile boundary is unable to transmit tension. The effect will be to loosen the top of the

pile as shown in Fig. 10a, and result in a hysteretic behaviour approximating that shown by the dashed lines in Fig. 10b rather than by the elasto-plastic behaviour indicated by the solid envelope. Hence in practice, the area within the real hysteresis loop will be somewhat less than half that of the elasto-plastic loop, and the damping, including the effect of soil yield, would have been about 7%. Thus it would appear unwise to rely on a significant reduction in response due to additional damping from soil yielding.

It is probable, however, that the effects of soil damping would have been more significant if the foundation compliance had taken the form of horizontal translation of the pile cap. As Fig. 11a shows, the single cylinder foundation results in a deflection profile where the soil displacement at ground level is small in comparison with the displacement of the centre of mass, and the foundation compliance effect results almost entirely from rotation of the pile cap. Under these circumstances soil damping is unlikely to be effective in reducing response. In Fig. 11b, in which the same centre of mass displacement from foundation compliance is now provided solely by translation of the pile cap, ground deformation will be greatly increased and soil damping may become a significant factor in reducing response.

CONCLUSIONS

Additional elastic flexibility in the form of foundation compliance was shown to theoretically result in large increases in curvature ductility factor demand if the displacement ductility factor for the bridge pier remained unchanged. Although dynamic analyses confirmed this result for bridge piers whose natural period was lower than that corresponding to peak response for the earthquake under consideration, the curvature ductility factor was comparatively insensitive to the increased elastic flexibility for longer period bridge piers. That is, increased elastic flexibility resulted in a reduced structure displacement ductility demand. It is possible that this effect results in part from increased hysteretic damping associated with the somewhat higher curvature ductility factors generally occurring under flexible base conditions.

The dynamic analyses indicate that current New Zealand design practice for ductility demand of bridge piers is very conservative for medium to long period structures, particularly when foundation compliance effects are considered, but may be slightly non-conservative for short period bridge piers.

Analysis of a typical single-cylinder foundation supporting a single-stem bridge pier indicated that additional damping provided by soil yield was unlikely to have a significant effect in reducing dynamic response. However, where foundation compliance takes the form of mainly translation rather than rotation of the pile cap, the reduction might be expected to be more significant.

Further research is needed to investigate the general effects of foundation compliance, and to provide more realistic modelling of soil yield.

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TABLE 1 : PROPERTIES OF PIERS

Pier Type	A	B	C	D
Stem Arrangement	Double-stem	Single-stem	Single-stem	Single-stem
Length L of pier* (m)	2.463	4.566	7.390	12.085
Natural Period, seconds**	0.2	0.5	1.0	2.0
Yield moment at plastic hinge, (kNm)	1,086	3,905	4,045	5,264
Plastic hinge length, (m)	0.292	0.488	0.956	1.266
Bilinear factor for moment-curvature curve, R	0.0384	0.032	0.0384	0.0384

* Distance from point of contraflexure to maximum moment.

** Calculated assuming rigid foundations.

TABLE 2 : DUCTILITY FACTOR DEMAND OF PIERS WITH FLEXIBLE FOUNDATIONS

Pier Type	Period, Earthquake Record	Maximum Curvature Ductility Factor, ϕ_u/ϕ_y				Maximum Overall Displacement Ductility Factor, μ			
		C = 1	2	3	4	C = 1	2	3	4
A	Natural Period, Sec.	0.20	0.30	0.38	0.44	0.20	0.30	0.38	0.44
	Base Shear Coeff. = 0.216g	El Centro N-S 1940 Taft B 69 W 1952 Bucharest N-S 1977 Artificial B1	31.40 17.09 12.67 49.92	57.81 41.60 15.69 121.74	70.51 81.05 31.62	11.97 6.45 5.17 18.74	11.99 8.28 3.93 22.52	10.28 5.41	8.5
	Calculated * Requirements $\frac{L}{P} = 0.118$	16.25	31.50	46.76	62.01	6	6	6	6
	Natural Period, Sec.	0.50	0.67	0.86	0.98	0.50	0.67	0.86	0.98
	Base Shear Coeff. = 0.209g	El Centro N-S 1940 Taft N 69 W 1952 Bucharest N-S 1977 Artificial B1	8.86 2.68 6.44 16.57	8.75 2.25 11.59 8.58	6.04 9.47 20.00	3.57 1.51 2.73 6.10	2.29 1.08 2.78 2.14	1.70 3.57	1.82
B	Calculated * Requirements $\frac{L}{P} = 0.107$	17.75	29.7	38.8	46.9	6	5.28	4.51	4.11
	Natural Period, Sec.	1.00	1.36	1.72	1.97	1.00	1.36	1.72	1.97
	Base Shear Coeff. = 0.137g	El Centro N-S 1940 Taft N 69 W 1952 Bucharest N-S 1977 Artificial B1 Artificial A1	7.47 1.02 18.76 8.23 15.57	3.48 1.44 21.19 9.01 11.87	1.95 3.45 20.40	3.57 1.02 8.19 3.93 6.61	1.49 1.05 5.32 2.66 3.31	1.13 4.1	1.25
	Calculated * Requirements $\frac{L}{P} = 0.129$	14.91	22.3	33.9	44.8	6	4.73	4.73	4.73
	Natural Period, Sec.	2.00	2.89	3.55	4.11	2.00	2.89	3.55	4.11
C	Base Shear Coeff. = 0.108g	El Centro N-S 1940 Taft N 69 W 1952 Bucharest N-S 1977 Artificial B1	4.03 Elastic 7.98 3.41	2.29 Elastic 6.05 5.48	3.75 Elastic 5.84	2.25 Elastic 3.78 1.93	1.44 Elastic 2.05 1.94	1.36 1.70	Elastic
	Calculated * Requirements $\frac{L}{P} = 0.105$	15.22	29.44	43.67	57.8	6	6	6	6

* μ from Ref. 2, and ϕ_u/ϕ_y from μ and Eqs. 4 and 7.

FIGURE 1: INFLUENCE OF ELASTIC DEFLECTION COMPONENTS ON PLASTIC ROTATION

FIGURE 2: COMPUTER MODELS OF BRIDGE PIERS

FIGURE 3: 'EXTENDED LEG' REPRESENTATION OF FOUNDATION FLEXIBILITY

FIGURE 4: EFFECT OF PERIOD SHIFT ON RESPONSE

FIGURE 5: COMPUTER MODELS TO INVESTIGATE SIGNIFICANCE OF PLASTIC HINGE LOCATION (TYPE C PIERS)

FIGURE 6: DISPLACEMENT RECORDS OF BRIDGE PIERS IN FIGURE 5

Deck consists of 7" thick slab on 4'-0" deep p.s.c. beams

(a) BRIDGE ELEVATION

(b) PIER ELEVATION

(c) SECTION THROUGH PIER

FIGURE 7: MOUNTAIN STREAM BRIDGE

FIGURE 8: MOUNTAIN STREAM BRIDGE PILE CAP AND REALISTIC COMPUTER MODEL

FIGURE 9: COMPARISON BETWEEN SOIL ELASTIC AND SOIL ELASTO-PLASTIC MODELS

FIGURE 10: COMPARISON BETWEEN ELASTO-PLASTIC AND EXPECTED SOIL HYSTERESIS LOOPS

FIGURE 11: INFLUENCE OF PILE CONFIGURATION ON FOUNDATION COMPLIANCE