Analytical Modeling of Spread Footing Foundations for Seismic Analysis of Bridges

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The way bridges respond to seismic excitation may be significantly influenced by the dynamic properties of their foundations. Within current design practice, foundation elements typically are considered as elastic springs, without consideration to material and radiation damping. General foundation models are identified that are suitable for (a) modeling soil-structure interaction for the seismic analysis of bridges, (b) modifying an existing, nonlinear, seismic-bridge-analysis computer program to include a new element capable of representing such models, and (c) conducting a parametric study to assess the effect of the increased energy dissipation mechanisms on the seismic response of bridge substructures. Three different models for spread-footing foundations are identified, applied to a typical two-column bridge bent, and compared with conventional elastic and fixed-base models. Three soil-stiffness values are considered, and two earthquake records, each with two different intensities, were applied to the bent. Maximum values of displacement, plastic-hinge rotation, and cumulative plastic hinge rotations were noted and compared. It was concluded that the use of the spread-footing foundation models can produce an important change in the bridge response to seismic activity when compared with that of the fixed-base model—depending on the frequency content of the earthquake and the stiffness of the soil. The effects of radiation damping were observed to be insignificant for foundations on stiff soil but important for those on soft soil. In addition, the performance of the simpler, damped foundation models was found to be quite similar to that of the more complex models. The models' accuracy was not verified, but the structural response of incorporating them was explored.

The way bridges respond to seismic excitation may be significantly influenced by the dynamic characteristics of the foundation (*1–3*). For example, interaction of the bridge superstructure with the abutments has been the cause of significant damage in past earthquakes (*3,4*). Although damage to other foundation elements, such as spread footings and piles, has been shown to be minimal, their performance during seismic excitation can have an important effect on the structural behavior (*5*), especially when the founding soil is soft (*6*).

Although research has shown that a significant amount of seismic energy is dissipated through the material and radiation damping associated with bridge supports and surrounding soil (*7*), these soil-structure interaction effects are not considered in detail in current design practice (*8*), and little emphasis has been placed on studying the role of foundations in the seismic analysis of bridges (*3,9*). Current design guidance is simplistic in that it considers the foundation elements as linear springs (*3,10*). The effects of gaps and the material nonlinearity of soil at abutments are approximated by manually varying the spring constants, such that the soil strength is not exceeded. However, important additional nonlinearities at abutments result from the force developed in the abutment key (*2*) and the energy loss due to impact during expansion-joint gap closure (*11*). Barenberg and Foutch (*12*) have reported that the elastic method is unconservative for abutments.

The role of foundations in seismic analysis is typically recognized through the use of translational and rotational springs. However, nonlinearities can arise from several sources, such as inelastic soil behavior and connection details at pile caps (*5*). Other important considerations include soil stiffness degradation that occurs during cyclic loading (*13*), loss of strength in the soil due to liquefaction, the influence of pile group behavior, and radiation damping. In addition, hysteretic damping may be included intentionally through the use of base-isolation techniques (*14–16*).

In order to properly represent hysteretic material damping and viscous radiation damping, Spyrakos (*8*) has recommended that a general, nonlinear, spring-damper model be used to represent the translational and rotational properties of piles, footings, and abutments. However, most computer software that is available for the dynamic analysis of bridges has only the capability to perform elastic analyses. Energy dissipation analysis is done through proportional damping, whereby a damping coefficient is associated with certain modes of vibration. Concentrated dampers and hysteretic springs, such as those that would be required to accurately model foundations, are not available for this type of analysis.

Nonlinear Earthquake Analysis of Bridge Systems (NEABS) (*17*) is a public-domain dynamic bridge analysis program that is capable of modeling nonlinearities. An algorithm for plastic-hinge formation and a gap-contact element are included in the program. However, there is no concentrated translational or rotational viscous damping element available for foundation modeling, nor is there a provision for stiffness degradation or strain hardening.

In this paper, the modification of the computer program, NEABS, to include discrete dampers and hysteretic springs for foundation modeling is described. The modified version of NEABS is then used to evaluate the effect of various foundation models and soil stiffnesses on the seismic response of a typical bridge bent founded on spread footings.

BACKGROUND

Soil-structure interaction refers to the effect that the founding soil has on the dynamic response of a structure and, conversely, the

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effect the structure has on soil motion. The structural response often includes an amplification of the translational motion, the introduction of a rocking component for an embedded foundation, an increase in the flexibility of the system, and the addition of damping from hysteretic action of the soil (hysteretic damping) and radiation of energy away from the structure in the form of outward-propagating soil waves (radiation damping).

Two general approaches are available for rationally incorporating soil-structure interaction effects into structural analysis (*18*). In the "direct method," the structure and a portion of the founding soil are both incorporated into a finite element mesh. This is the simplest approach conceptually, but a number of drawbacks, including the need for a large model, energy-absorbing boundaries, and detailed soil properties, make its use prohibitive for all but the most extreme cases.

A simpler, more efficient approach is the substructure method. Here, the structure and the soil are analyzed separately. A simplified model is constructed that can approximate the behavior of the soil at the foundation. This simplified model is then coupled with the structure at the supports, and the structure is analyzed.

The foundation model typically is composed of one or more springs or spring/damper combinations arranged in series or kept parallel for each degree of freedom. The combinations are chosen on the basis of the assumed foundation behavior, which is obtained either experimentally or analytically.

The most common analytical model is one in which the soil domain is considered to be a homogeneous, elastic half-space. The frequency domain solution for the dynamic response of a rigid disk on an elastic half-space has been derived and extended for footings of various other shapes and depths of embedment. One should note that the disk/half-space solution is frequency dependent. For nonlinear dynamic analysis, which must be conducted in the time domain, various foundation models have been proposed that reproduce the analytical foundation response for certain ranges of loading frequencies. Four such models, consisting of combinations of linear springs, masses, and dampers, are shown in Figure 1. For a comprehensive review, one may refer to works by Wolf (*19*) and Richart et al. (*20*).

MODIFICATION OF NEABS

The computer program NEABS was chosen as the means to implement the methods that have been proposed to include the effects of soil-structure interaction in bridge analysis. The source coding for NEABS is in the public domain and it was obtained and modified. In order to apply the models mentioned above to represent the dynamic properties of bridge foundations, a new, discrete foundation element was added—a parallel combination of a spring and viscous damper.

Description of NEABS

NEABS originally was developed by Tseng and Penzien in 1973 to study the seismic performance of long, multiple-span bridges

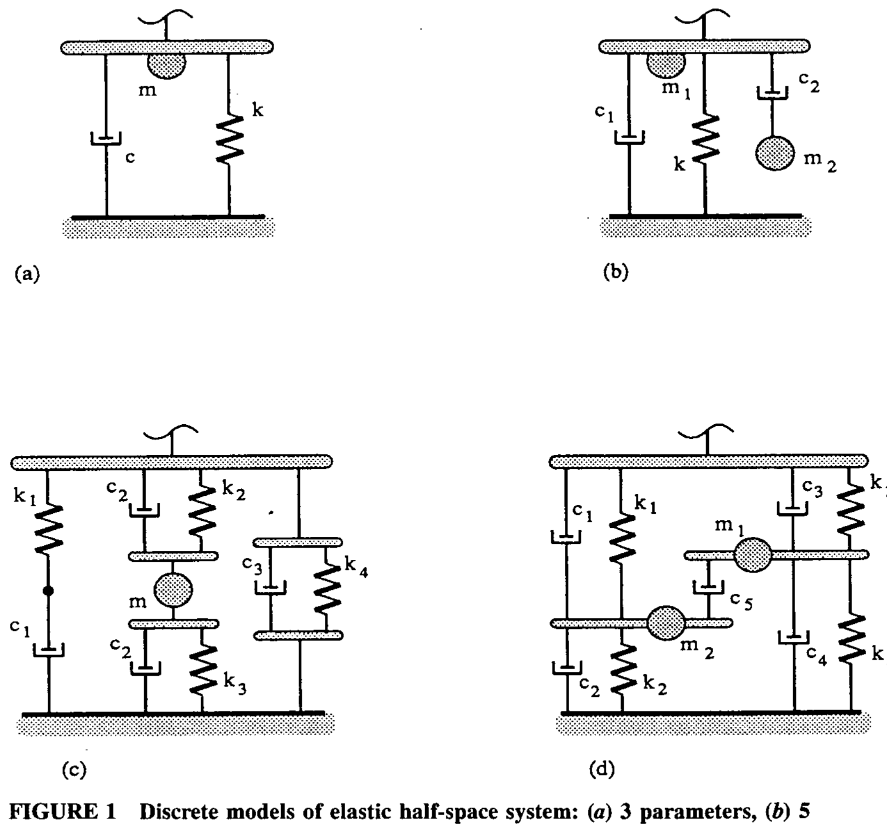


FIGURE 1 Discrete models of elastic half-space system: (*a*) 3 parameters, (*b*) 5 parameters, (*c*) 9 parameters, and (*d*) 11 parameters.

(*21*). Using the finite element method, NEABS idealizes a structure as a discrete system subject to nodal dynamic loadings or prescribed support motions.

Four element types are available to model the structural members of a bridge. Deck sections and columns are modeled with a beam element that may be either elastic or elasto-plastic. In the case of the elasto-plastic beam, the ends are allowed to develop perfectly plastic hinges. An elastic curved beam element is also available. Supports may be given elastic stiffnesses with a boundary spring element. A nonlinear expansion-joint element is included that can model the opening and closing of the joint gap, the impact at gap closure, and elasto-plastic joint tie bars.

Lumped masses and mass moments of inertia may be assigned to structure nodes directly or may be specified through mass densities for both the straight and curved beam elements. Energy dissipation not included as yielding in the elasto-plastic elements is accomplished globally by using two-parameter Rayleigh viscous damping. With Rayleigh damping, the global damping matrix is assumed to be a linear combination of the global mass and stiffness matrices. For an elastic structure, this has the effect of assigning a unique damping ratio to each of the structure's modes of vibration.

Both static and dynamic nodal loadings may be prescribed, as can support motion. Dynamic nodal loads and support motions are specified by supplying load and acceleration-time histories, respectively.

The equations of motion are solved in the time domain to allow nonlinear response, using the Newmark method of direct time integration. Either constant or linear acceleration between time steps may be assumed. At each time step, the out-of-balance force vector from the previous time step is added to the current applied equivalent force to minimize the accumulation of integration errors. In addition, the program will iterate and subdivide the time step used in the integration to ensure that the Euclidean norm of the out-of-balance force vector is within prescribed tolerances. Output consists of both the forces and displacements of the initial static response and time histories of the dynamic response. These time histories may consist of nodal displacements, nodal accelerations, member forces, and, for nonlinear elements, member-nonlinear (plastic) displacements.

Discrete Foundation Element

As previously discussed, the foundation models for soil-structure interaction may range in complexity from simple, linear spring supports to those employing a number of internal nodes, masses, dampers, and nonlinear springs. Accordingly, the Discrete Foundation (DF) element was formulated as a general purpose element to enhance the capabilities of NEABS. The element connects two nodes, which may actually occupy the same location, as in a simple foundation model.

The DF element is a parallel combination of a spring and viscous damper. Thus, to model the more complex systems shown in Figure 1, several DF elements and internal foundation nodes are required. For example, model (*d*) in Figure 1 would require five DF elements and two internal nodes. Note that the DF element used to model *c5* would include damping and zero stiffness.

The model built with DF elements connects the base of the structure element, for example, a column, and a fixed support. Separate properties are used for each of six local degrees of freedom, and there is no stiffness or damping coupling. Mass and mass moments of inertia may be lumped at each end node, including internal foundation nodes, and each degree of freedom, independently.

The DF element spring stiffness is bilinear to allow elasto-plastic behavior and hysteretic material damping. Kinematic strain hardening is incorporated as the default, but isotropic hardening or a combination of the two may be specified. A gap and stiffness degradation, as a function of deformation, may also be included.

The damping coefficients for each DF element may be specified separately for all degrees of freedom, allowing discrete dampers to be included in a foundation model. This damping is independent of the Rayleigh viscous damping in that the contribution of the DF element to the global mass and stiffness matrix is not considered when determining the Rayleigh contribution to the global damping matrix. Thus, the Rayleigh damping concept may be used for the bridge structure without affecting the concentrated dampers present in the foundation models. A complete description of the DF element may be found elsewhere (*22*).

PARAMETRIC STUDY

A parametric study was undertaken to investigate the effects of incorporating foundation models of varying complexity into bridge seismic analysis. The purpose was to compare various foundation models with each other and with a fixed support to evaluate their effect on the structural response of a bridge bent. One should note that, since the study results were not correlated with experimental response data, the study does not constitute a verification test of these models' accuracy. Rather, it is an exploration of the structural response effects of incorporating these models in seismic bridge analysis. The foundation models are consistent with elastic half-space assumptions, as previously discussed. Establishing consistency between these assumptions and actual behavior is beyond the scope of this paper.

Description of the Model

An existing highway bridge was chosen to provide guidance for the development of the structural analysis model. A solitary bridge bent was modeled so that only the effects of the spread footing foundation, and not that of abutments, would be included.

The bent consisted of two 7.6 m long, 91 cm diameter reinforced concrete columns on spread footings, supporting a cross beam, which supported the bridge superstructure. The 107-cm wide, 91-cm deep cross beam was cast monolithically with the diaphragm and deck and, because the resulting composite assembly was quite stiff in comparison with the columns, the cross beam was assumed to be rigid. The bent was assumed to support a dead load of approximately 1050 KN. The centerlines of the two columns were 7.3 m apart. Longitudinal reinforcing bars were spaced evenly around the cross-section perimeter, and they extended into the crossbeam with no splice. The spread footing dimensions were 2.9 m square in plan and 61 cm deep. A schematic of the model analyzed is shown in Figure 2. Specific details of the bent are given elsewhere (*22,23*).

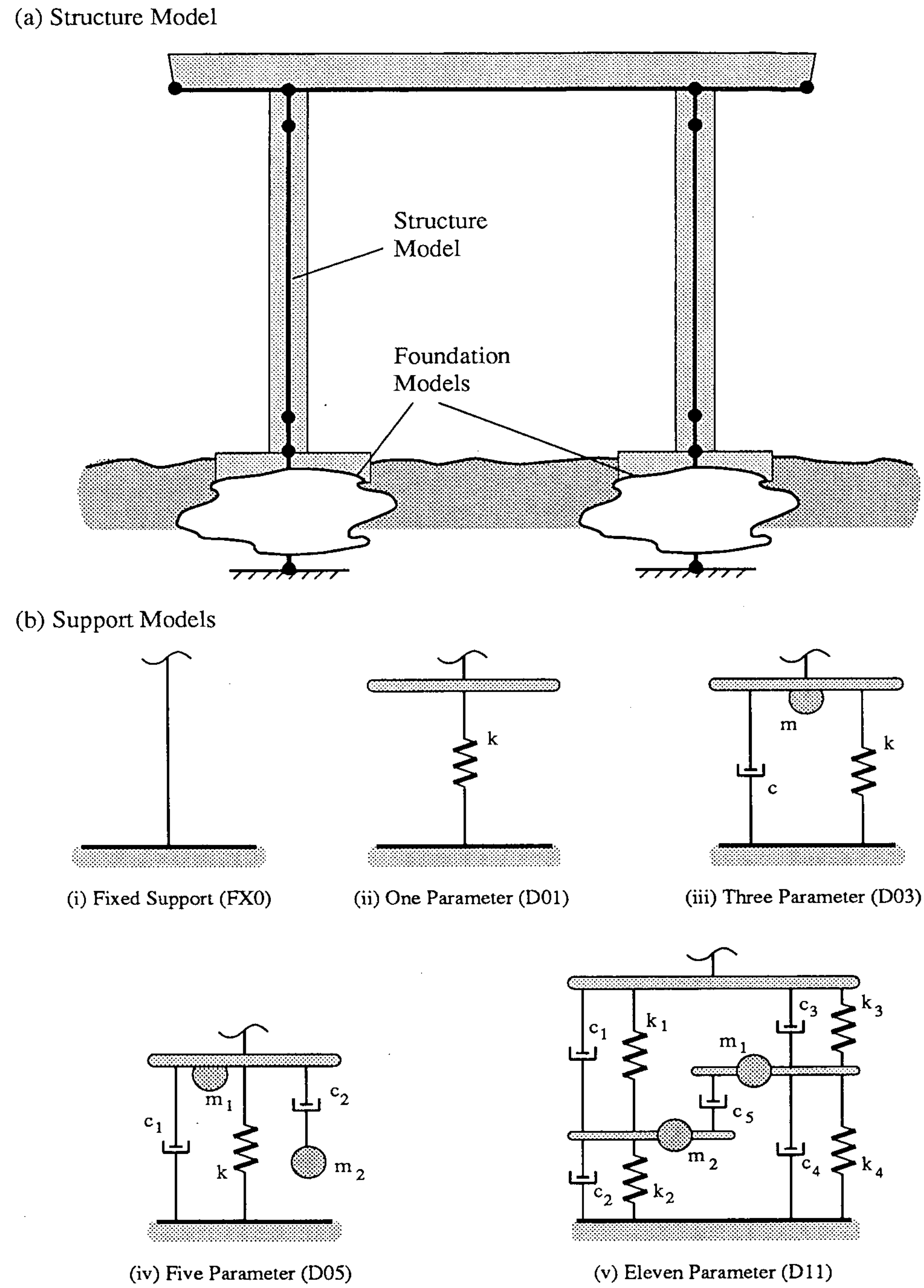


FIGURE 2 Schematic of NEABS models for the spread footing foundation study; (*a*) bent structure, and (*b*) foundation models.

The bent was modeled with nine beam elements and it was supported on the various foundation models, composed of DF elements. The foundation properties were assigned independently to the three planar degrees of freedom: horizontal translation, vertical translation, and rocking. All other degrees of freedom were constrained. The modulus of elasticity that was used for the columns was E = 31.7 GPa. The moment of inertia that was used was half that of the gross transformed column cross section, to account for the effect of initial concrete cracking. The yield surface for the elasto-plastic beam elements was based on the axial force-bending moment strength interaction curve. Rayleigh damping, corresponding to 5 percent of critical for the fundamental period of the fixed-base bent, was added to the structure.

Five foundation models were considered, as shown in Figure 2. One model consisted of fixed supports, one consisted of elastic supports, and three had damped elastic supports that required 3, 5, and 11 parameters per degree of freedom, respectively. All but the fixed support are discrete approximations of the elastic half-space continuum model, but with increasing levels of complexity. The footings were not assumed to be embedded. Because the half-space is elastic, the damping that is present in the foundation models corresponds to radiation damping only. Energy dissipation from material damping has not been quantified and, therefore, it is not included.

Three soil stiffness values were used in testing each model. The stiffnesses were selected to span a range of values commonly encountered. The unit weight of the soil was taken to be 10.8 kN/m^3. Three shear wave velocities, of 91.5, 213.5, and 396.5 m/sec, were chosen to produce the three soil stiffnesses. For the given soil density and the assumption of small strain, these corresponded to soil shear moduli, G, of 14.7, 80.3, and 277 MPa, respectively. Poisson's ratio for the soil was taken to be $\nu = 0.33$. The stiffness, mass, and damping values that were assigned to each foundation model are given in Table 1. Formulas for obtaining these values may be found elsewhere (*19,22,24,25*). The fundamental periods for the bent ranged from 0.53 sec for the fixed-base foundation to 0.68 sec for the most flexible foundation.

Recorded acceleration histories from actual earthquakes formed the basis of the seismic excitation applied to the bent-foundation system. The two earthquake records chosen were the S00E component of the El Centro record of the 1940 Imperial Valley Earthquake (referred to as the "El Centro" record) and the N86E component of the Olympia record of the 1949 Western Washington earthquake (or "Olympia" record). Acceleration history plots are given in Figure 3.

To incorporate variations in record intensity in the study, both records were scaled to an intensity of 0.25 g effective peak acceleration ("lower" intensity) and to an intensity of 0.40 g effective peak acceleration ("higher" intensity). The definition of effective peak acceleration is outlined in the recommendations of the National Earthquake Hazards Reduction Program (*26*).

Results

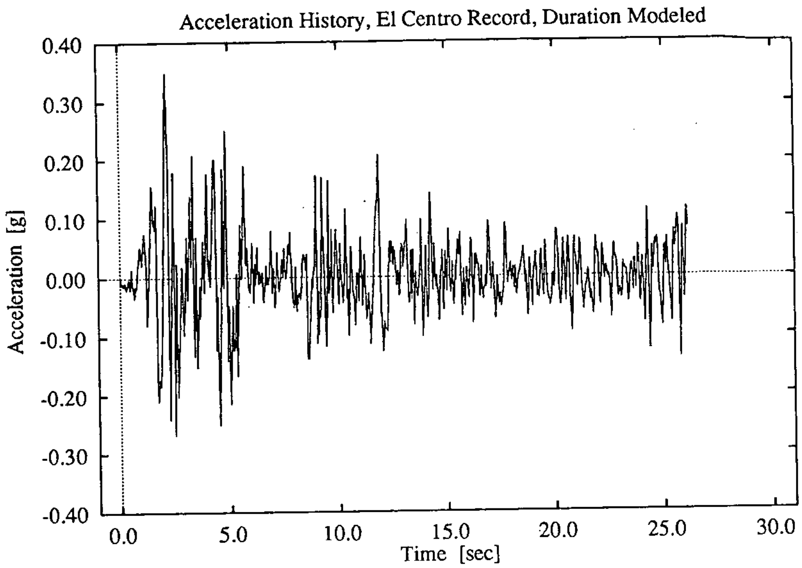
The performance of the various foundation models was assessed in terms of their effects on the response of the bent structure. Specifically, three aspects of the bent's response were selected to be studied: column displacement, that is, the displacement of the column top relative to the bottom, the moment at the top of the column, and the plastic-hinge rotation at the column top. This information was provided by the program in the form of time histories. The results were then interpreted in terms of their implications for column ductility demand and energy dissipation demands. One should note that the column moment values reported by NEABS include a dynamic component from damping in addition to the usual moment that results from stiffness.

A number of analyses were performed, consisting of five foundation models, three soil stiffness values, and four seismic input records. Four graphs of the data from each NEABS analysis were used, examples of which are shown in Figures 4 and 5. In Figure 4, the time histories of the column displacement and column moment for the higher intensity El Centro earthquake record, soft

TABLE 1 Parameter Values for Spread Footing Foundation Models

		Soft Soil				Intermediate Soil				Stiff Soil			
		D01	D03	D05	D11	D01	D03	D05	D11	D01	D03	D05	D11
Lateral Translation	k1	1.52E+05	1.52E+05	1.52E+05	1.04E+05	8.26E+05	8.26E+05	8.26E+05	5.64E+05	2.85E+06	2.85E+06	2.85E+06	1.94E+06
	k2				2.98E+05				1.62E+06				5.59E+06
	k3				1.00E+05				5.45E+05				1.88E+06
	k4				3.15E+05				1.71E+06				5.91E+06
	c1		2.06E+03	1.80E+03	2.30E+03		4.81E+03	4.20E+03	5.36E+03		8.94E+03	7.81E+03	9.95E+03
	c2			0.00E+00	6.31E+03			0.00E+00	1.47E+04			0.00E+00	2.73E+04
	c3				3.47E+02				8.10E+02				1.50E+03
	c4				4.33E+03				1.01E+04				1.88E+04
	c5				5.32E+03				1.24E+04				2.31E+04
	m1		7.92E+00	0.00E+00	9.23E+00		7.92E+00	0.00E+00	9.23E+00		7.92E+00	0.00E+00	9.23E+00
	m2			0.00E+00	5.21E+01			0.00E+00	5.21E+01			0.00E+00	5.21E+01
Vertical Translation	k1	1.72E+05	1.72E+05	1.72E+05	1.91E+05	9.37E+05	9.37E+05	9.37E+05	1.04E+06	3.23E+06	3.23E+06	3.23E+06	3.59E+06
	k2				2.44E+05				1.33E+06				4.58E+06
	k3				7.88E+04				4.29E+05				1.48E+06
	k4				3.11E+05				1.70E+06				5.85E+06
	c1		3.11E+03	2.45E+03	2.39E+03		7.27E+03	5.72E+03	5.58E+03		1.35E+04	1.06E+04	1.04E+04
	c2			8.23E+02	5.50E+03			1.91E+03	1.28E+04			3.54E+03	2.39E+04
	c3				1.28E+03				2.98E+03				5.54E+03
	c4				4.29E+03				1.00E+04				1.86E+04
	c5				4.90E+03				1.14E+04				2.12E+04
	m1		2.11E+01	0.00E+00	6.88E+01		2.11E+01	0.00E+00	6.88E+01		2.11E+01	0.00E+00	6.88E+01
	m2			6.04E+00	4.44E+01			6.04E+00	4.44E+01			6.04E+00	4.44E+01
Rocking	k1	3.39E+05	3.39E+05	3.39E+05	1.48E+05	1.85E+06	1.85E+06	1.85E+06	8.04E+05	9.87E+09	9.87E+09	9.87E+09	4.30E+09
	k2				5.73E+05				3.12E+06				1.67E+10
	k3				5.14E+05				2.80E+06				1.50E+10
	k4				3.68E+05				2.01E+06				1.07E+10
	c1		1.87E+03	0.00E+00	-1.90E+03		4.36E+03	0.00E+00	-4.43E+03		8.10E+03	0.00E+00	-8.22E+03
	c2			2.42E+03	7.28E+03			5.64E+03	1.70E+07			1.05E+04	3.15E+07
	c3				4.52E+03				1.05E+04				1.96E+04
	c4				3.57E+02				8.34E+02				1.55E+03
	c5				3.82E+03				8.91E+03				1.65E+04
	m1		3.13E+01	0.00E+00	1.39E+02		3.13E+01	0.00E+00	1.39E+02		3.13E+01	0.00E+00	1.39E+02
	m2			2.89E+01	4.43E+01			2.89E+01	4.43E+01			2.89E+01	4.43E+01

Note: unit of force = kN, unit of length = m, unit of rotation = radian, unit of time = sec.



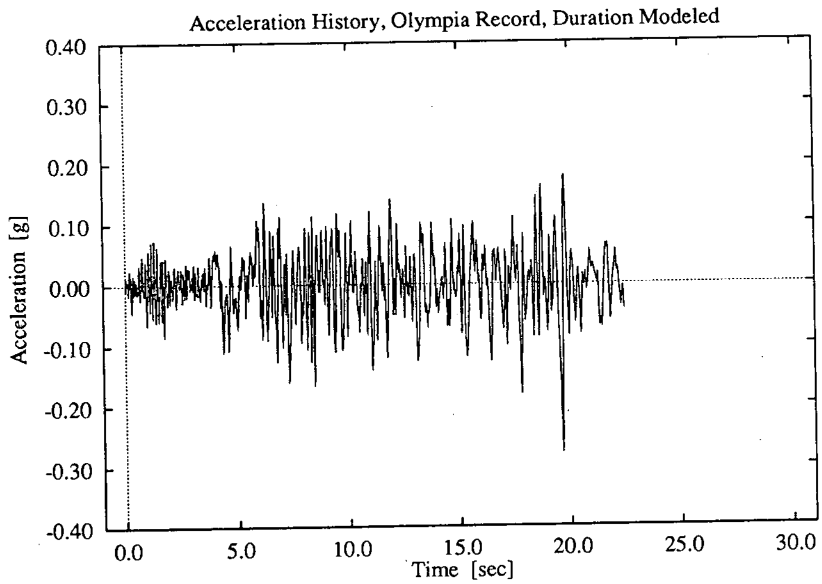


FIGURE 3 Earthquake acceleration history plots.

soil, and 11-parameter foundation model are given. The third graph, shown in Figure 5 for the same analysis, depicts the column moment-displacement hysteresis, which may be used as an indicator of energy dissipation demand. The fourth graph, also shown in Figure 5, is a time history of the plastic-hinge rotation at the top of the column.

Whereas the column remains elastic, the moment in the column does not produce plastic rotation; this condition results in a horizontal line in this graph. A vertical line indicates that a plastic hinge has formed at the column top, and it is being rotated by the moment. The magnitude of these plastic rotations is indicative of instantaneous ductility demand at the top of the column. Also, if the axial force on the columns is assumed to be constant, or nearly so, over the duration of the excitation, then the moment required to yield this column will also be constant. If this is the case, then work done on the plastic hinge over the excitation duration will be the yield moment multiplied by the sum of the absolute values of plastic rotation, represented by the vertical lengths on the graph. As the assumption of nearly constant axial force is reasonable, this graph can also provide an indication of the cumulative energy dissipation demand of the top of the column.

To summarize and compare these results, the maximum plastic rotation (measured from the undeformed state) and the sum of all

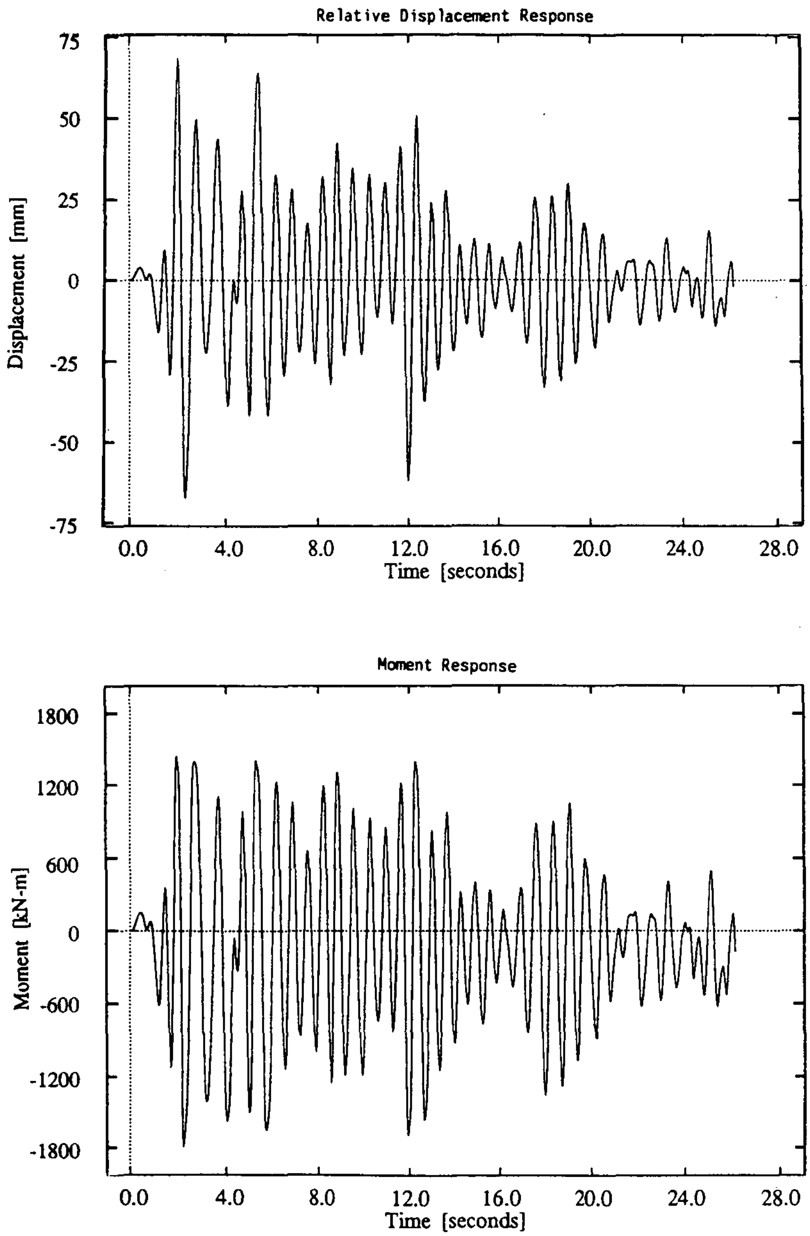


FIGURE 4 Typical time history results.

plastic rotation was calculated for each run. As mentioned, these quantities are related to ductility and energy dissipation demands. These data are given in Figures 6 through 9. Each figure shows a set of bar charts of both the rotation maxima and rotation sums for the given excitation record. Each bar chart shows the results of the four discrete foundation models for each soil stiffness, and allows a comparison with the fixed-support results. In Figure 2, a schematic of each foundation model is shown.

Discussion of Findings

The response of the bridge bent to the two earthquake records is somewhat different, although the intensity of each earthquake resulted in plastic-hinge formation for almost all analyses. For both El Centro records, the stiff and intermediate foundation models led to nearly the same instantaneous and cumulative demands as those of the fixed-base model. The soft foundation model resulted in a significant increase in cumulative demand for both intensities, and it led to increased instantaneous demand for the lower intensity record. The instantaneous demand for the higher intensity El Centro record was approximately the same for all of the foundation models.

The flexible foundation caused an increase rather than a reduction in column demand. By comparing the earthquake record of Figure 3 with the example plastic-hinge rotation history of Figure 5, one may observe that much of the damage results from peak

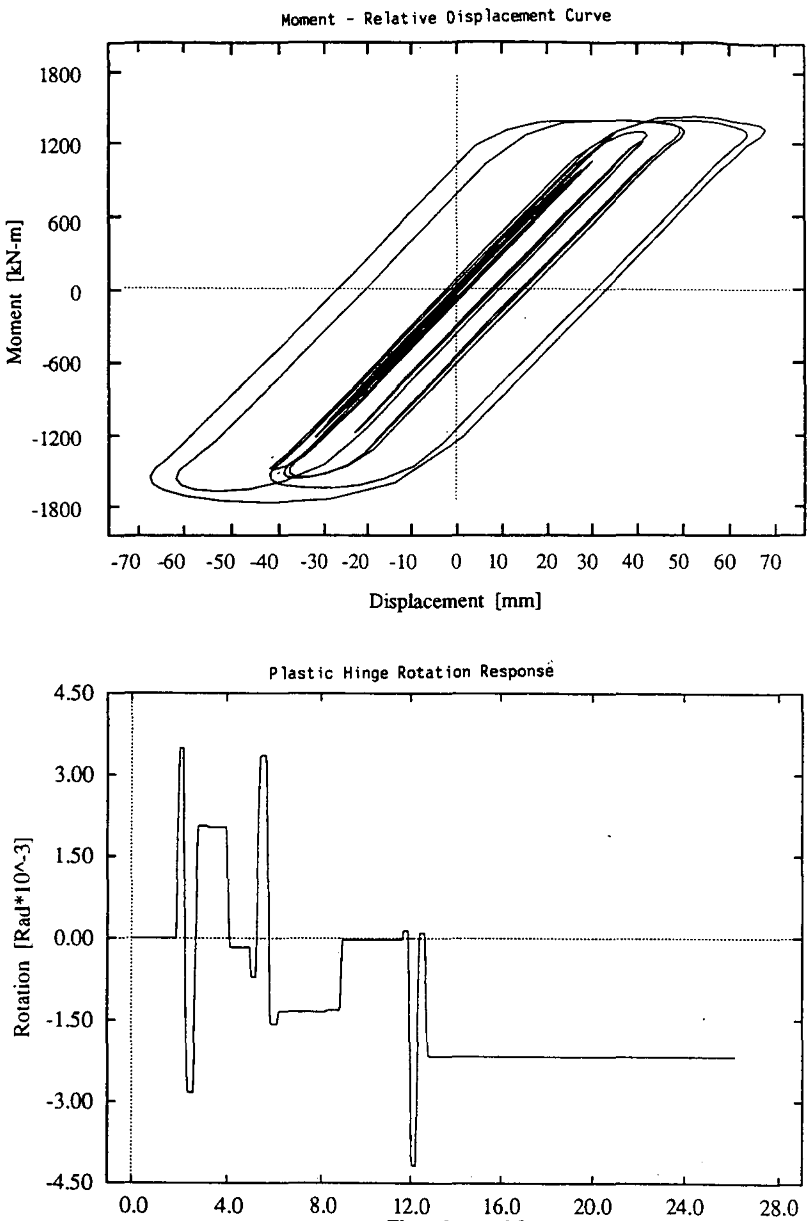
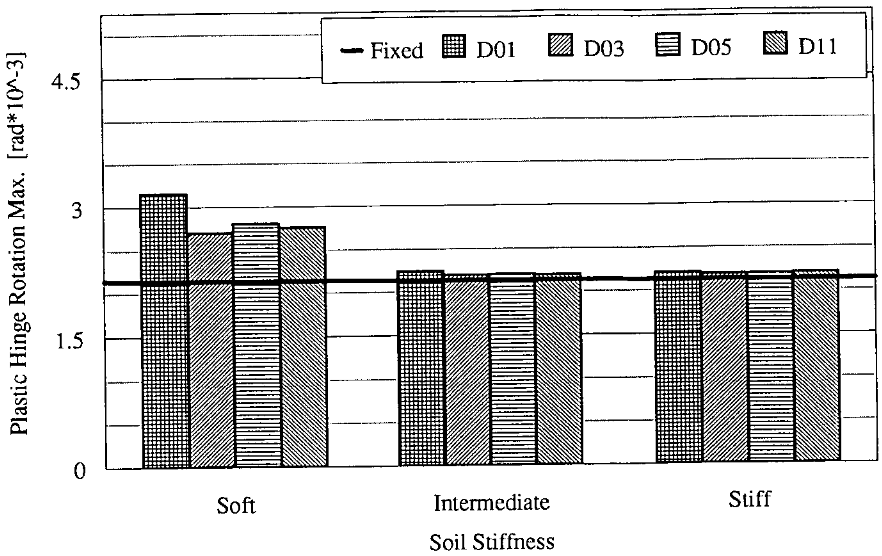


FIGURE 5 Typical hysteresis and plastic hinge rotation results.

accelerations at approximately 2 sec, 5 sec, and 12 sec. The pulse at 12 sec seems to be the major source of the increase in demand over the other foundations because its period of application is close to the fundamental period of the structure with the flexible foundation.

Damping in the discrete foundation model had a negligible effect on the column demands for the intermediate and stiff foundations. However, the damped foundations (the 3-, 5-, and 11-parameter models) caused a reduction in demand in the order of 15 to 20 percent, when compared to the spring foundation alone, for the soft soil. Also, little change was observed between the simple and more complex damped models. This is likely due to the fact that the damping and mass values for the three-parameter model are relatively insensitive to the loading frequency for translational motion, which seemed to dominate the response.

For the Olympia earthquake records, the instantaneous demands on the column were of the same order as those of the El Centro records for the intermediate and stiff foundation models, but much less for the soft foundation model. Indeed, no column yielding was indicated for the lower intensity Olympia record and the soft foundation. This appears to be the result of the frequency content of the earthquake versus the natural frequencies of the structure-foundation system.



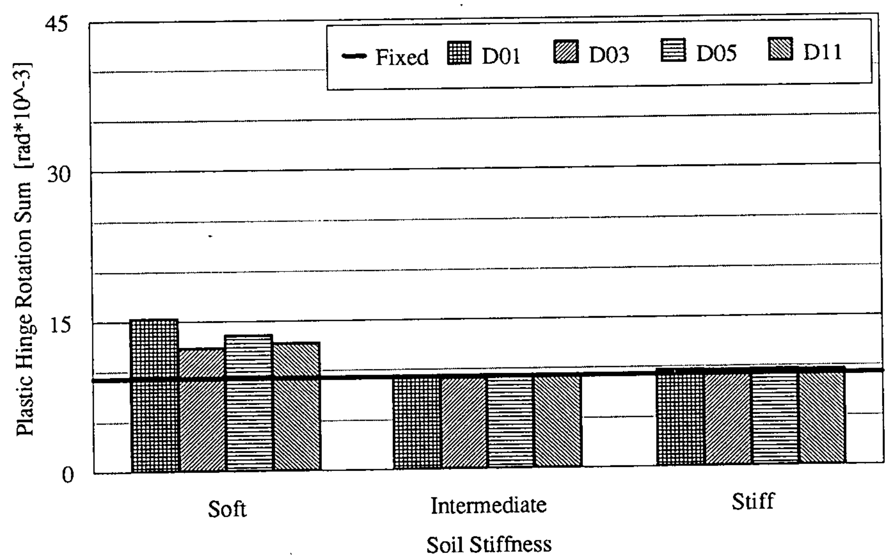


FIGURE 6 Comparison of instantaneous and cumulative column demands, lower intensity El Centro earthquake record, spread footing foundation.

The cumulative demand, however, was significantly less for all foundations when compared to that of the El Centro earthquake. The two earthquake records were scaled to the same effective peak accelerations, but, from Figure 3, it is apparent that the Olympia record is dominated by a single peak at approximately 20 sec. Because the majority of the column damage is caused by this peak, as opposed to several different peaks in the El Centro record, the total amount of plastic-hinge rotation is reduced.

As with the El Centro earthquake, radiation damping was significant only for the column cumulative demand and the soft foundation. The reduction in demand from damping ranged from approximately 25 to 35 percent.

CONCLUSIONS

A new and versatile foundation element has been developed and implemented into the nonlinear, dynamic, bridge analysis program, NEABS. Because of its ability to include concentrated dampers and bilinear springs with strain hardening, stiffness degradation, and a gap algorithm, the new element can be used to model the behavior of various types of bridge supports, including footings, elastomeric bearing pads, base isolation devices, piles, and abutments. Here, a parametric study was performed to investigate the effect of different foundation models and soil types on

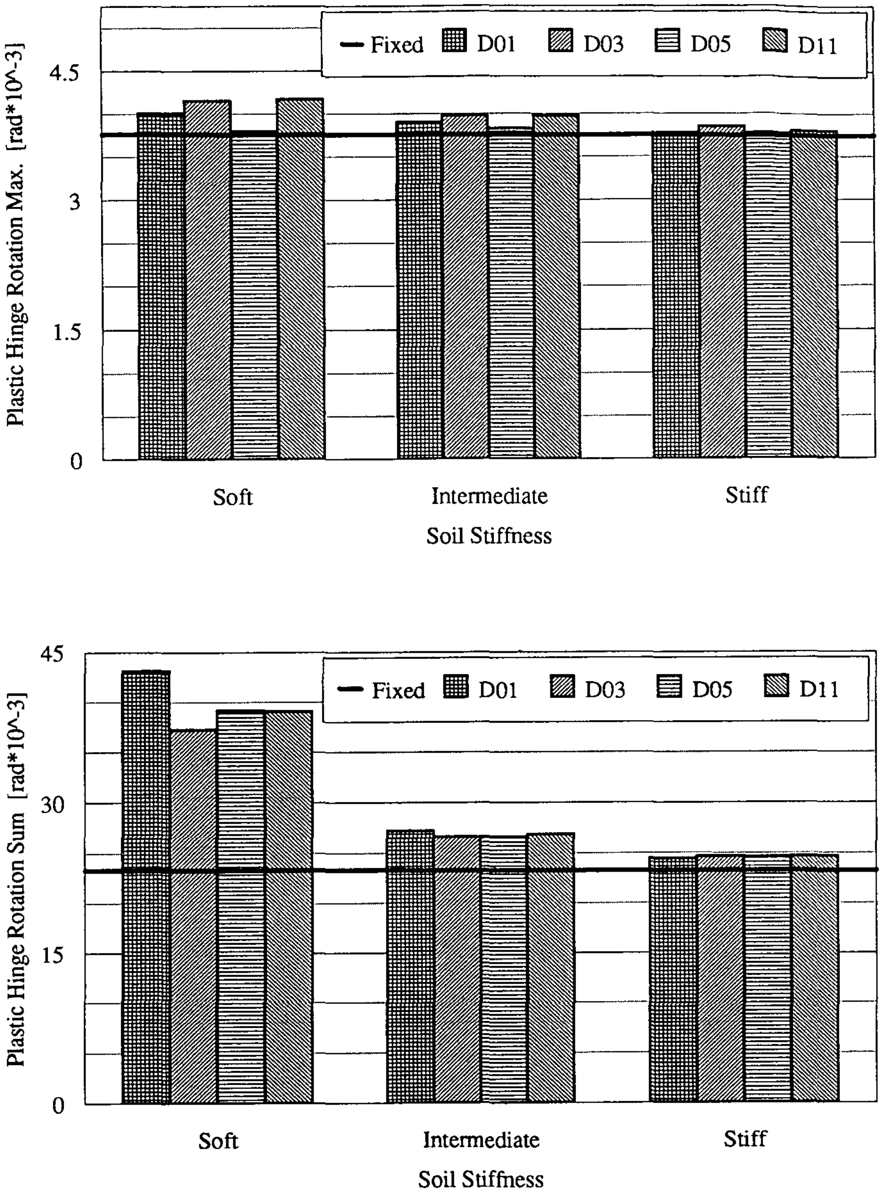


FIGURE 7 Comparison of instantaneous and cumulative column demands, higher intensity El Centro earthquake record, spread footing foundation.

the response to earthquake excitation of a bridge bent on spread footings. From the results, several conclusions may be drawn.

The enhancement of a fixed-base model to include foundation flexibility has a dramatic influence on the column demands during strong earthquakes. This seems to be a result of variations in the natural frequencies of the system, and the actual effect depends on the frequency content of the earthquake. For the El Centro records, increased column demands were noted for the flexible foundation, whereas, for the Olympia records, the intermediate foundation was critical. Thus, no conclusion can be drawn regarding whether one foundation is more critical than another. However, the results indicate that a fixed-base model could easily underpredict column demands for an earthquake analysis. One should note that, in order to evaluate the effect of foundation properties on bridge response in a consistent manner, no attempt was made to alter the earthquake records on the basis of an assumed soil layer. To include such effects, a separate analysis to obtain free field motion at the site must be performed.

The addition of concentrated dampers to model radiation damping had a significant effect only when the foundation was soft. As expected, the energy absorption of the dampers acted to reduce cumulative demand on the columns. Neglecting the radiation damping would probably have little effect on the response of similar structures when founded on soil of high or intermediate stiff-

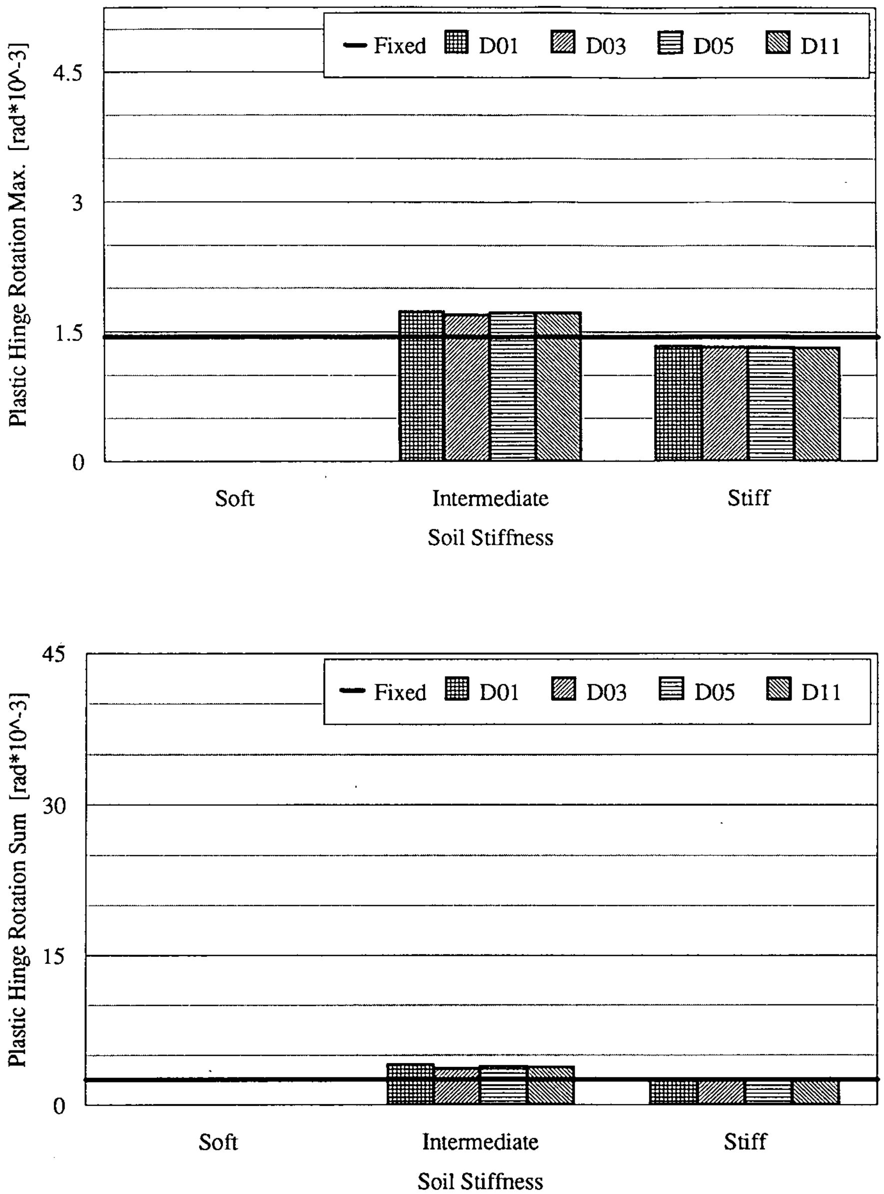


FIGURE 8 Comparison of instantaneous and cumulative column demands, lower intensity Olympia earthquake record, spread footing foundation.

ness. For soil of low stiffness, however, the use of elastic foundations alone could lead to a somewhat conservative prediction of inelastic demand. If damping is added, the simpler, three-parameter model produced results that were in close agreement with those of the more complex models.

The foundation models were based upon the assumptions of elastic half-space theory. Refinements to the theory, including solutions for a layered half-space and a viscoelastic half-space, have been proposed. Hysteretic action around the supports and gap behavior could be modeled by employing a nonlinear near-field element in series with a far-field element based on half-space theory, such as those we have described. Near-field properties must be defined for specific foundation types, however, such as piles and abutments. These are items for further research.

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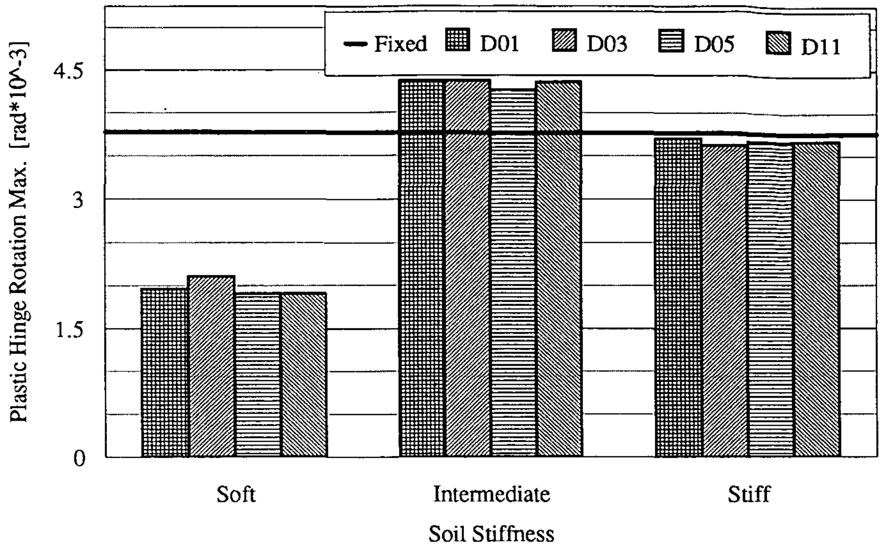

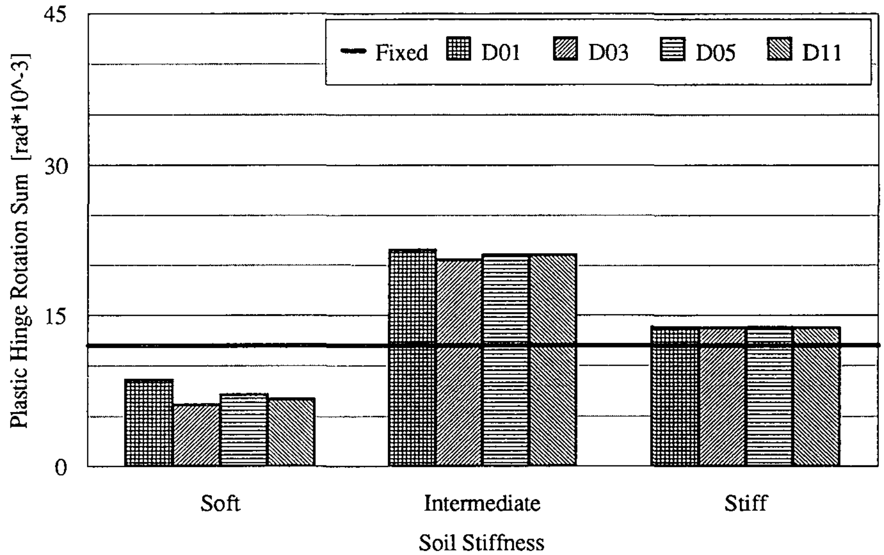

FIGURE 9 Comparison of instantaneous and cumulative column demands, higher intensity Olympia earthquake record, spread footing foundation.

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