APPROXIMATING INELASTIC RESPONSE OF STRUCTURES TO GROUND SHAKING

by

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SYNOPSIS

Inelastic response of structures to ground shaking can be approximated by a practical analytical alternative that is more sophisticated than a typical elastic analysis but less complex than an inelastic time-history procedure. Data from two real structures and actual recorded motion are used to illustrate the alternative procedure. Elastic and inelastic capacities of the structures are estimated and are reconciled by graphical methods with the demand response spectra of ground shaking events. The solution results in values for peak structural response, peak ductility demands, equivalent response periods of vibration and percentages of critical damping, and reserve capacities.

INTRODUCTION

When a structure is subjected to severe ground shaking -- such as that caused by a large earthquake that causes lateral forces substantially greater than the lateral force design criteria -- portions of the structure are expected to exceed their elastic limit and respond in a nonlinear, inelastic manner. In the course of this inelastic response, the apparent or effective periods of vibrations and percentages of critical damping will vary from the values expected during an elastic response. This is due to the softening effect or stiffness reduction and the hysteretic or energy-absorption changes caused by the inelastic action of some of the building elements. Complex time-history computer programs can simulate the inelastic characteristics of these elements, but these programs are expensive, time-consuming, and depend on assumptions of inelastic properties that cannot always be well defined. Also, a time-history solution is valid for only one particular set of building properties and a ground motion history. Any change in parameters requires an additional computer analysis. Whereas this type of analysis may be justified for some large projects that have sufficient funds and technical resources available, it would be difficult to justify its use on most projects done by the average practicing engineer. Another approach to providing for the expected inelastic response of a structure to ground shaking is by means of an elastic analysis. Force factors or coefficients that take into account the ductility and reserve strength of the structure are applied to the lateral force design. Thus, it is assumed that the demands of a large earthquake will be satisfied if the elastic capacity of the structure is exceeded. This approach is the basis for most existing seismic design requirements.

This paper presents an alternative to the procedures described above, filling a gap between the complex inelastic time-history procedure and the less sophisticated standard elastic analysis procedure. The capacity of the structure is determined by combining an elastic analysis with some gen-

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eral bilinear approximations. The demand of the ground shaking is represented by response spectra at two or more values of critical damping. Capacity and demand are reconciled by a graphical solution that accounts for changes in both the apparent response periods of vibration and percentages of critical damping.

DESCRIPTION OF PROCEDURE

The procedure requires the determination of two curves, one representing the capacity of the structure and the other representing the demand of the ground shaking, described here by spectral acceleration (S_a) and response periods of vibration (T) (other terms, such as spectral displacements, roof displacements, and base shear coefficients, could also be used). Only the fundamental mode of vibration will be considered, although the effects of the higher modes can be estimated.

The capacity characteristics of the structure are determined in much the same way as in the reserve energy technique[1] -- either by simple hand methods or by more complex computer analysis methods, depending on the complexity of the structure and the accuracy required. First, the elastic capacity threshold[2] is determined in terms of spectral acceleration, spectral displacement, and fundamental period of vibration. A mathematical model is developed that best represents the structure at this amplitude of lateral motion. Periods and participation factors are calculated. The lateral force that causes a substantial number of major members to yield is determined. The amplitude of force may be represented by a base-shear coefficient, a lateral roof displacement, or a lateral roof acceleration. These values can then be converted to spectral values by using the participation factors. Next, the characteristics of the structure beyond the elastic range are estimated. A new mathematical model is developed; it is similar to the elastic model except that all the yielding members are assigned stiffness properties that are greatly reduced. For example, if all the girders on several or all the floors are assumed to be yielding, the moments of inertia of these girders might be reduced to 5% of the elastic values in order to approximate a bilinear effect. For this new mathematical model, a set of periods and participation factors are calculated, and the lateral force that is required to cause a more extreme failure condition is determined. This failure condition may be due to additional members yielding, members exceeding their ductility capacity, brittle failures, excessive displacements, or instability. Several intermediate thresholds may be determined depending on the conditions of the problem. Each step is represented by segmental values of period of vibration, spectral acceleration (ΔS_a), and spectral displacement (ΔS_d). Figure 1 plots spectral acceleration and spectral displacement values, that are somewhat equivalent to a force (represented by acceleration) versus displacement curve, where the slope represents the stiffness of the structure. The cumulative values of spectral accelerations (S_a) and spectral displacements (S_d) can be used to calculate an effective period of vibration (T_{eff}) for the multilinear system by using Equation (1).

$$T_{eff} = 2\pi\sqrt{\frac{S_d}{S_a\ (g)}} \tag{1}$$

Figure 2 plots the effective period and spectral acceleration values.

The demand characteristics of the ground shaking are represented by response spectra. These spectra can either be standard shapes scaled to the site, spectra developed especially for the site, or spectra obtained from recorded ground shaking. At least two values of damping are required, one representing the elastic structure, and the other representing the structure at its maximum inelastic excursions. It is assumed that effective damping varies somewhat linearly between these two conditions.

Having established the capacity characteristics and the demand characteristics, the two sets of data are plotted on the same graph and their intersection is considered to be the reconciliation between demand and capacity,[3] as shown in Figures 3 and 4.

EXAMPLES

Sources of data used in the examples are from studies of two Holiday Inn structures that responded to the San Fernando earthquake of 1971,[4,5] and two 4-story reinforced concrete test structures that have been subjected to ground motion caused by underground nuclear explosions at the Nevada Test Site.[2,6,7] Measured response of both sets of structures indicated that the elastic capacities of their structural systems had been exceeded during their response to severe lateral motion. Measured response data are also available for these structures for ambient and low-amplitude excitation both before and after the severe lateral motion.

The two almost identical Holiday Inn structures are 7-story reinforced concrete frame structures. One is located in Van Nuys, California, about 13 miles from the epicenter of the 1971 earthquake, and the other is located in Los Angeles, California about 26 miles from the epicenter. Peak recorded ground accelerations were in the range of 15% to 25% of gravity, and both buildings were damaged by the earthquake. Repairs, primarily nonstructural, were about 11% of the initial construction cost for the Van Nuys building and 7% for the Los Angeles building. The structures were mathematically modeled from data on the architectural and structural drawings. Gross concrete sectional properties were used with some allowance for floor slab participation and column reinforcement. The results of the structural analysis indicated an elastic limit threshold at a lateral roof displacement of 2.4 cm in the transverse direction and 2.1 cm in the longitudinal direction. At the elastic limit thresholds defined above, all columns were stressed well below their elastic limit capacities, although a significant number of beams were stressed beyond their elastic limit capacities. It was clear from the analysis that all beams would yield before any columns would yield. To obtain an approximation of the characteristics of the structure for lateral displacement beyond the elastic limit threshold, a supplemental mathematical model was analyzed, where all beams were assigned stiffness values equal to 5% of the values used in the original elastic model. An inelastic lateral displacement capacity equal to about ten times the elastic limit displacement was assumed, based on the gross concrete sections used in the models, the structural details of the beam-column joints and their ductility capacities, the strength capacities of the columns, maximum interstory displacements, and the absence of any potential collapse mechanisms. The elastic and inelastic values are combined to estimate the resultant values. The resultant periods were calculated from Equation (1) using the sum of the spectral accelerations and displacements. Table 1 summarizes the capacity characteristics of the elastic, inelastic, and combined models of the transverse direction of the structure. The demand characteristics are represented by the response spectra of the actual recorded ground level motion of the 1971 earthquake. It is assumed that the structures respond at 2% of critical damping prior to reaching the elastic limit threshold, and that damping will increase to 10% of critical at the far limits of the inelastic response. The demand and capacity characteristics for the transverse direction of the two structures are plotted on Figures 3 and 4. The predicted response of the structures to the actual recorded motion is obtained from the intersection of the two curves. In this example, the amplitudes of motion are slightly less and the periods are slightly longer than the actual recorded values. However, the capacities were based on the bare structural frames, and a more detailed investigation[4], which considered the participation of nonstructural elements[8,9] showed better agreement with the recorded results. The point along the inelastic portion of the capacity curve that is intersected by the demand curve gives an indication of the ductility demand and the reserve capacity. If the coordinates (S_a, T) are converted to a value of S_d using Equation (1), the calculated spectral displacement value can be compared to the elastic limit value (Table 1) for ductility and to the combined elastic-inelastic value for reserve capacity.

The two Nevada Test Site structures, which were constructed in 1965-1966, are 4-story slab and frame structures supported by four columns. They are 12 ft by 20 ft in plan and 36 ft in height. These structures have been subjected to a continuing program of vibration testing, some of which is the subject of another paper at this conference.[7] During the eight years before the high-amplitude tests of 1974,[7] the measured fundamental periods ranged from about 0.4 to 0.5 sec, with the variations depending on the amplitude of motion and the previous response history.[6] The peak measured lateral roof displacements during this time were between 2 and 3 cm. Preparation for the 1974 tests included predictions of the response characteristics of the structure at damaging amplitudes of motion. Using a procedure similar to the one used for the Holiday Inn structures, the periods were estimated to be 0.5 sec at the elastic limit and about 0.9 sec at a peak lateral roof displacement of about 11 cm. These estimates were fairly close to the results of the destructive testing done in 1974.[7]

CONCLUSIONS

The described procedure is proposed as a reasonable approximation of inelastic response of structures to earthquake-like ground motion. Results of the Holiday Inn and 4-story test structure analyses indicate that the procedure is reasonably reliable. The examples illustrate that although the demand spectral acceleration for the elastic model of the structure may greatly exceed the elastic capacity, the structure can survive the earthquake motion due to the inelastic response characteristics.

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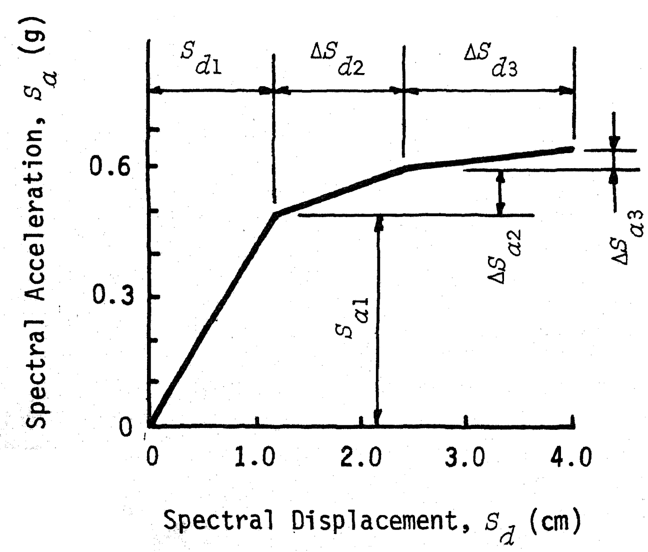

FIGURE 1 SPECTRAL ACCELERATION VERSUS SPECTRAL DISPLACEMENT

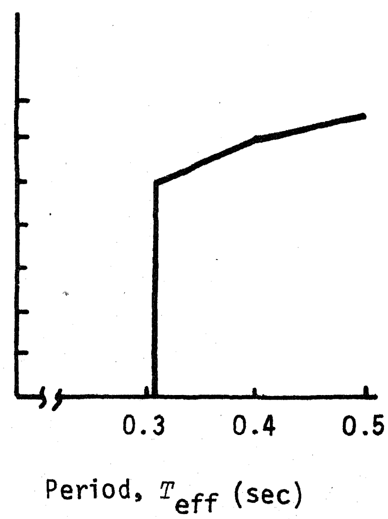

FIGURE 2 SPECTRAL ACCELERATION VERSUS EFFECTIVE PERIOD

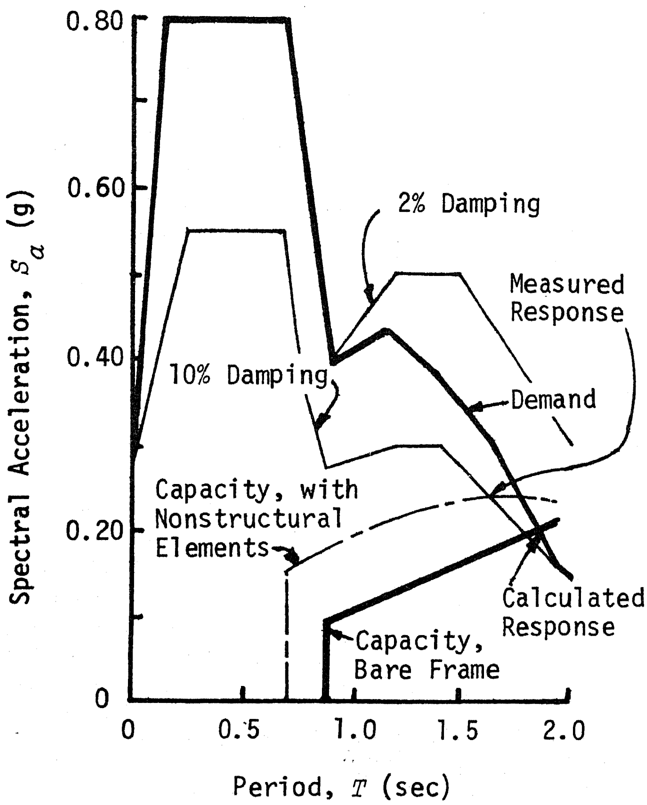

FIGURE 3 CAPACITY VERSUS DEMAND, VAN NUYS, TRANSVERSE DIRECTION

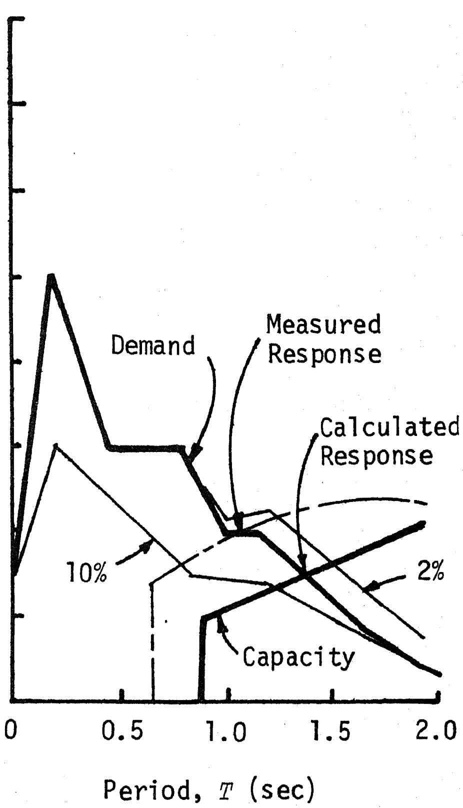

FIGURE 4 CAPACITY VERSUS DEMAND, LOS ANGELES, TRANSVERSE DIRECTION

TABLE I

HOLIDAY INN SUMMARY FOR TRANSVERSE DIRECTION

	Elastic Model	Inelastic Model	Combined Model
Lateral Displacement at Roof, cm	2.4	24.0	26.4
Period, sec	0.88	2.49	1.93
Ratio Roof Displacement to Spectral Displacement	1.31	1.40	--
Spectral Displacement, cm	1.83	17.14	18.97
Spectral Acceleration, g	0.095	0.111	0.206