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A CASE STUDY FROM THE NORTHRIDGE EARTHQUAKE

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ABSTRACT

Measurements of high ground accelerations in the vertical direction during the Northridge Earthquake of Jan. 17, 1994 point to the possible need for evaluating the effect of this component of ground motion on the seismic response of highway bridges. This paper presents the results of a study of the I-5/SR-14 Separation and Overhead Bridge which failed during the Northridge Earthquake. The results show that the seismic forces due to vertical component of ground motion could have caused hinging in the superstructure. The shear demand associated with hinging of the superstructure exceeds the shear capacity of the superstructure resulting in brittle shear failure. The postulated failure scenario based on this mode of failure is remarkably similar to the observed damage. Design implications and the need for future research are discussed.

KEYWORDS

Vertical Motion; Bridges; Seismic; Northridge Earthquake; Shear Failure.

INTRODUCTION

During the last two to three decades, many studies, both analytical and experimental, have been performed to provide a better understanding of the behavior of highway bridges under seismic loading. Most of these work have concentrated on substructures, especially the columns and piers due to the fact that most of these studies have considered only the effect of horizontal motions. Superstructures are typically very stiff and strong in the horizontal directions and they do remain elastic when subjected to seismic excitations in these directions. Another major factor is that based on current design methodology, the columns and piers are the dominant source of energy dissipation through nonlinear behavior, thus, emphasis has been placed on evaluation of seismic response of this component of a bridge system. For the same reason the few work in which the effect of vertical motion on the seismic response of highway bridges has been investigated have also concentrated on the nonlinear behavior of the columns and piers. This paper presents an analytical case study of a prestressed bridge subjected to vertical earthquake motion. It is shown that within typical ranges of input ground accelerations and dynamic amplification factors, it is possible to cause the formation of plastic hinges in the superstructure, and that the shear demand to cause hinging will exceed the shear capacity of the superstructure causing brittle failure. The results of this study and the inferred mode of failure are consistent with the observed damage in I-5/SR-14 Separation and Overhead which failed during the Northridge Earthquake of Jan. 17, 1994.

DESCRIPTION OF I-5/I-14 SOUTH OVERHEAD AND SUSTAINED DAMAGE

Description of the Bridge

This is a 10-span structure with five frames connected to each other through expansion joints. The general elevation and plan of the bridge are shown in Figure 1. The ends and central frames have cast-in-place prestressed box girder superstructures. The other two frames (i.e. frames 2 and 4) have reinforced concrete box girder superstructures. The depth of the superstructure is 7′, and the hinge seat width is 14″. The superstructure is supported on elastomeric bearing pads on 3′ wide seat abutments at each end. The abutments are founded on spread footings. All bents consist of single rectangular columns of 12′X4′ to 12′X6′ which flare to a 26′ width at the top and extend into the ground as a 12′ circular shaft. The four hinges of this bridge were retrofitted with cable restrainers as one of the first retrofit projects following the 1971 earthquake.

Description of Damage

The collapsed frame (i.e. frame 1) consists of a seat abutment and two single column bents (continuous spans over these bents), and seats at the right end on an expansion joint (EJ) near the first bent of the next frame. Features of sustained damage (Caltrans, 1994, EERC, 1994, EERI, 1994, Priestley *et al.*, 1994) include: total disintegration of the column in bent 2, flexural cracks at the bottom of the girder at the location of bent 2, movement of the top of the column at bent 2 (about several feet toward bent 3), splitting of the bent cap and apparent punch through of the column at bent 3, damage to the soffit of the hinge, unseating of the superstructure at the abutment (the deck was lying on the ground about 5′ from the face of the abutment), failure of one of the shear keys, failure of cable restrainers, and stirrups were ripped from the cap beam at bent 3. It is reported that there was no evidence of any longitudinal movement at the abutment backwall and that the backwall remained intact with no signs of cracking (Caltrans, 1994). This does not support the possibility of longitudinal excitation of frame 1 as the cause of failure.

DYNAMIC ANALYSIS OF THE BRIDGE

Analytical Model

Frame 1 (i.e. the collapsed frame) was analyzed using linear as well as nonlinear frame models to determine the dynamic characteristics in the vertical direction and to evaluate the level of seismic forces that can be generated under the vertical component of the ground motion. The analytical model consists of linear elastic beam elements representing the columns and the superstructure, and a nonlinear spring element to model the seat abutment. This part of the study was performed using the finite element program ADINA (1994).

The nonlinear spring element does not have any resistance in the upward direction (i.e. the deck can separate from the abutment once gravity load is overcome). In the downward direction a high stiffness is assigned to the spring constant. The expansion joint end of the frame is assumed pinned. The columns are fixed at their base. Note that due to high stiffness of the superstructure, the vertical modes of vibrations are not sensitive to the length and boundary conditions at the base of the columns. Figure 2 shows the analytical model along with the geometry and material properties used in this study.

Dynamic Characteristics

Due to nonlinear boundary condition at the abutment, the mode shapes, frequencies and participation factors will be different in the downward direction from those in the upward direction. Therefore, to determine the dynamic properties of the system two linear cases are considered. In one case the abutment is assumed pinned and in the second case the abutment is assumed to be free. The former will correspond to motion in the downward direction and the later to motion in the upward direction. In the downward direction the response is dominated by only one mode shape which is shown in Figure 3. The dominant mode shapes, frequencies, and participation factors for upward motion are shown in Figure 4. As it can be seen from these figures, the dynamic response of the system in the upward direction will be significantly different than its response in the downward direction.

Vibration of the bridge in the downward direction will mean higher forces in the bridge deck, although they will be in the direction of gravity (direction of design forces). Therefore, the possibility of damage to the deck will depend on the inherent factor of safety in the design. This depends on several factors such as the ratio of dead load to live load, type of construction (e.g. prestressed vs. reinforced concrete), type of material used, etc. For example, for an old reinforced concrete bridge this factor may be very low since the ratio of dead load to live load is higher in older bridges. On the other hand, use of high strength concrete and higher design loads (heavier trucks) means that new reinforced concrete bridges will have a higher reserved capacity.

Vibration of the system in the upward direction will exert forces in the system that will be in the opposite direction of the design forces. For example, as the right span moves in the upward direction positive moment will be developed at the column bent in the superstructure. This is in the opposite direction of the design moments, and the capacity of the deck cross section in this direction can easily be exceeded by moderate level of excitation, especially for prestressed concrete bridges. Note that the upward mode shapes will be effective when the gravity load of the bridge (i.e. dead load) is overcome.

Time History Analysis

Using the analytical model discussed along with the free field vertical component of Newhall recordings (with PGA of 0.62g) from the Northridge Earthquake (Darragh *et al.*, 1994), a time history analysis was performed to determine the level of seismic forces that can be generated in the system under vertical excitation. The I-5/SR-14 connector is located only several miles away from the location of Newhall recording. Since cracking in the deck is the only source of damping in the vertical direction and the superstructure is prestressed a damping ratio of 3% was assumed.

Time histories of the moments in the superstructure at the right end of left span and the left end of right span are shown in Figure 5. Note the difference in the frequency content of the time histories for these two sections. This is due to directionality of the mode shapes as discussed before, which has an effect on the response of the left span. However, it does not significantly affect the response of the right span. The maximum moments at critical sections are given in Table 1. For comparison these values corresponding to dead load are also given.

Table 1. Moments at Various Sections

Section	Max. Moments from Time History Analysis		Dead Load
	Downward motion ft-kips	Upward motion ft-kips	Moment ft-kips
R-LS	-123,950.	+28,900.	-49,562.
L-CS	-127,067.	+18,967.	-53,316.
R-CS	-127,114.	+32,527.	-52,312.
L-RS	-133,453.	+45,223.	-48,917.

R = right, L = left, LS = left span, CS = center span, RS = right span

CAPACITY OF THE SUPERSTRUCTURE

Description of the Prestressing & Material Properties

The total jacking force specified for frame 1 is 13,400 kips and the total area of the prestressing tendons is 55 in^2 (in each web there are six tendons each consisting of 0.5 in diameter strands). The center of gravity of the prestressing force was 10″ from the top over bents 2 and 3, and at midspans from the bottom of the deck it was: 23″ at the right span, 9″ at the center span, and 23″ at the left span.

Material properties for the bridge used in moment-curvature analyses and determination of shear capacities are: f'_c = 6,000 psi, f_y = 60 ksi, f_{pu} = 300 ksi, where f'_c is compressive strength of concrete, f_y yield stress for non-prestressed reinforcements, and f_{pu} is the ultimate strength of the prestressing tendons. The rupture strain for prestressing tendons is assumed to be 0.04 in/in. Note that the plans specify 28-day strength of 3,500 psi, however, it is expected that the actual maximum compressive strength is as high as 6,000 psi (Priestley *et al.*, 1994).

Moment-curvatures analyses were performed using the RESPONSE computer program by Collins and Mitchell (1991). The program assumes a Ramberg-Osgood type of stress-strain relationship for the prestressing tendons. For concrete a parabolic function was employed to define the stress-strain relationships.

Moment-Curvature Relationships

The maximum flexural strength for various sections are given in Table 2. Because of the location of prestressing tendons the ultimate strength of the superstructure at any section under positive and negative moments are significantly different from each other. Evaluation of moment curvature diagrams for various cross sections demonstrate that they posses high ductility (in the range of 8 to 14).

By comparing moment capacity of the deck to the results of the time history analysis (i.e., Table 1) it is obvious that moments in the superstructure due to vertical excitation of the bridge will exceed the ultimate moment capacities in both upward and downward directions. In the downward direction at the left end of the right span the moment capacity is exceeded by a factor of 1.5. In the upward direction the moment capacity of the deck is exceeded by a factor as high as 5.2, again at the left end of the right span. Thus, even a much smaller input ground motion would cause development of plastic hinges. If the shear demand corresponding to a given plastic hinge mechanism is greater than the available shear capacity of the deck brittle shear failure will follow.

Shear Capacity

The shear capacity for various sections is determined using ACI 318-89 guidelines (1989). ACI considers the fact that two types of shear cracking can occur in concrete beams, namely web-shear cracking and flexural-shear cracking. Therefore, the shear strength provided by the concrete for a prestressed section is the lesser of the shear required corresponding to these two situations.

Shear strength corresponding to flexural-shear cracking can be computed using the following equation (ACI, 1989):

$$V_{ci} = 0.6\sqrt{f'_c}b_w d + V_d + \frac{V_i M_{cr}}{M_{max}}$$

That is, for a noncomposite uniformly loaded beam the shear strength provided by concrete is equal to $0.6\sqrt{f'_c}b_w d$, plus the shear force required to generate the cracking moment in the section. Note that V_{ci} need not be taken less than $1.7\sqrt{f'_c}b_w d$.

Under the web-shear cracking assumption, the concrete shear strength is (ACI, 1989):

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3f_{pc}\right)b_w d + V_p$$

In these equations: f'_c is the maximum compressive strength of concrete; b_w is the web width; d is the distance from the extreme compression fiber to the centroid of the prestressed reinforcement (but not less than 0.8h, where h is overall cross section depth); V_d shear due to unfactored dead load, M_{max} maximum factored moment, V_i shear corresponding to the maximum factored moment at the section; M_{cr} moment causing flexural cracking due to the externally applied load; f_{pc} compressive stress in the concrete at the centroid of cross-section resisting externally applied loads; and V_p vertical component of effective prestressed force at the section.

The nominal shear strength provided by the shear reinforcement is $V_S = \frac{A_v f_y d}{s}$ where A_v, f_y, and s are area, yield stress, and spacing of the shear reinforcement. #5 stirrups were used as shear reinforcement where the spacing was 10″ at the end spans adjacent to the column bents, and where it was 8″ at the center span adjacent to the column bents.

Using these equations the nominal shear capacity for various sections for both upward and downward directions were calculated and are given in Table 2. The yield stress for the shear reinforcements is 60 ksi. The cracking moments were calculated assuming a rupture of modulus of $6.0\sqrt{f'_c}$. For both directions, the shear strength provided by the concrete is controlled by the flexural-shear cracking.

Table 2. Superstructure Moment & Shear Capacity at Various Sections

Section	Moment		Shear	
	Positive ft-kips	Negative ft-kips	Upward kips	Downward kips
R-LS L-RS	8,684	-87,795	845 (637)	3962 (2310)
L-CS R-CS	8,684	-87,795	845 (637)	5062 (2310)
Center of Midle Span	95,124	-18,024	nc	nc
Center of End Spans	77,156	-34,286	nc	nc

The number in parentheses is the shear strength provided by the concrete
R = right, L = left, LS = left span, CS = center span, RS = right span, nc = not critical

POSSIBLE CAUSE OF FAILURE

Plastic Analysis of the Bridge:

Plastic analysis of the bridge was performed to determine the shear demand corresponding to various plastic hinging mechanisms. It was determined that plastic hinging in the upward direction controls as discussed below.

In the upward direction the first plastic hinge will form at the right end of the left span. Based on the time history analyses the maximum moment at this section is 28,900 ft-kips, while the moment capacity is only 8,684 ft-kips. The shear required is 104 kips which is much smaller than the available shear capacity of 845 kips. The second plastic hinge will form at the left end of the center span and the third hinge at its right end. The fourth plastic hinge will form at the left end of the right span (i.e. right of bent 3). The ultimate capacity of the section at this point is also equal to 8,684 ft-kips. Both center and right spans now have hinges at their ends and act like simply supported beams independent of each other. Formation of a plastic hinge at the center of any one of these spans will cause a flexural mechanism. The maximum shear at the center span corresponding to such a mechanism is 513 kips which is smaller than the shear capacity of the superstructure at 845 kips. However, the shear required at the left end of the right span (i.e. at the right of bent 3) to cause a mechanism is 1,098 kips. This is higher than the available shear capacity of 845 kips.

Interestingly, in the downward direction the maximum shear required for plastic hinging is 3,895 kips, which is just slightly less than the provided capacity of 3,962 kips.

Thus, before formation of the third plastic hinge in the right span the bridge will fail in a brittle manner as the shear capacity of the superstructure at the juncture of the right span and the column at bent 3 is exceeded.

Proposed Failure Scenario Due to Vertical Motion:

Considering the results of time history analysis and ultimate analysis of the superstructure, the following failure scenario under vertical motion is proposed:

i. Due to the vertical ground motion and excitation of the bridge in this direction, it is possible to develop moments as high as five times the ultimate capacity of the cross section. Thus, plastic hinges will develop in the superstructure.

ii. The shear capacity of the superstructure at the juncture of bent 3 is smaller than the shear required to develop plastic hinging mechanism. Consequently, the superstructure fails under shear at the right of bent 3. The upward shear forces pull the stirrups from the bent cap which in turn pull the entire bent cap concrete. That is, since the stirrups in the bent cap are U shaped, they will scoop out the bent cap concrete as one end of the U gets pulled by the deck (in this case the end on the side of the right span) in the upward direction. However, the concrete covering the other side of the U-shaped stirrups will remain intact. This is consistent with the observed damage at the bent cap which includes total disintegration of the cap beam and stirrups ripped from the bent cap, while the right side (i.e. the center span face) of the cap beam is practically intact.

iii. Due to brittle shear failure at bent 3, the right span (i.e. from bent 3 to the expansion joint) falls off causing tensile failure of the restrainers at the expansion joint. Again this is consistent with reported damage observations.

iv. Due to the weight of the falling span (about 2400 kips) and release of prestressing forces (over 10,000 kips), a lateral force through the prestressing tendons and toward bent 3 (north) will be applied to the remaining portion of the frame. Since the integrity of the bent cap is lost no force will be transferred to the column at bent 3. This lateral force easily exceeds the shear capacity of the column at bent 2 and friction force at the abutment. Thus, consistent with the observed damage, the left and center spans are pulled toward bent 3 (in the northern direction) causing unseating of the superstructure at the abutment and total disintegration of the column at bent 2, while the column at bent 3 survives.

v. As the deck is mobilized and moves toward bent 3 (north), it also slides toward east on the abutment. This is due to the fact that the deck is tilted toward east because of the curvature of the road in the longitudinal direction. Thus, the superstructure hits and damages the shear key at the east side as it slides off the abutment.

Note that based on the time history analysis the maximum shear at the left end of the right span is 1,619 kips and the maximum moment in the upward direction at the left end is 45,223. ft-kips. Thus, with a maximum shear capacity of only 845 kips and a plastic moment capacity of 8,684 ft-kips at the left end, a much smaller peak ground acceleration (about 0.4g) would cause the proposed mode of failure. Three recording stations in the vicinity of the bridge discussed measured peak vertical ground acceleration of 0.59g or higher.

It is interesting to note that the shear capacities of the other two prestressed frames of this bridge (i.e. frames 3 and 5), which did not fail, are larger than the shear forces required to cause plastic hinging mechanisms. This is due to the fact that for these frames the centroid of the prestressing tendons at the center of the spans is located at 9″ from the bottom. This results in much smaller moment capacity in the upward direction (i.e. smaller negative moment capacity at the center of each span). Thus, similar to the center span of the frame discussed, the shear required to cause a plastic hinging mechanism will be smaller. Of course, other factors such as spatial variations of the ground motion and dynamic characteristics of each frame have contributed to the lack of damage to these frames. For example, it is reported that the canyon in which the I-5/SR-14 Separation and Overhead is located may have experienced large spatial variations of ground motion (EERC, 1994).

CONCLUSIONS

Based on the results of this study the following conclusions are made and areas for future research are identified:

i. It is quite possible that the failure of I-5/SR-14 Separation and Overhead during the Northridge Earthquake of Jan. 17, 1994 was caused by vertical vibration. Failure scenario based on this mode of failure is remarkably similar to the observed damage in the bridge.

ii. It may be very well that restraining the expansion joints in the vertical direction, as a result of seismic retrofitting, contributed to this mode of failure. Restraining the vertical movement results in higher shear demand to cause plastic hinging in the upward direction. Of course, from a flexural failure point of view installing cable restrainers is beneficial. This is an area that requires further research.

iii. There appears to be a need to evaluate the seismic performance of bridges in the vertical direction, especially for those near seismic faults. As a minimum and the first step, the shear capacity of the superstructure must be higher than the shear required to develop plastic hinging mechanisms in both upward and downward directions

iv. The effective shear area for prestressed superstructures in the upward direction is not known and requires further research. Both experimental and analytical research are needed to determine the shear capacity when the deck, due to upward motion, is subjected to reversed state of loading (i.e. moment arm is h-d rather than d).

v. In general, prestressed concrete cross sections, similar to the one considered in this study, demonstrate good curvature ductilities. However, due to the unsymmetric nature of the moment-curvature curves, the relationship between the local ductility and global ductility demand is not known. Furthermore, smaller effective depth in the upward direction will certainly mean much smaller plastic hinge length which in turn will significantly affect the global ductility supply. This is another area that also needs further research.

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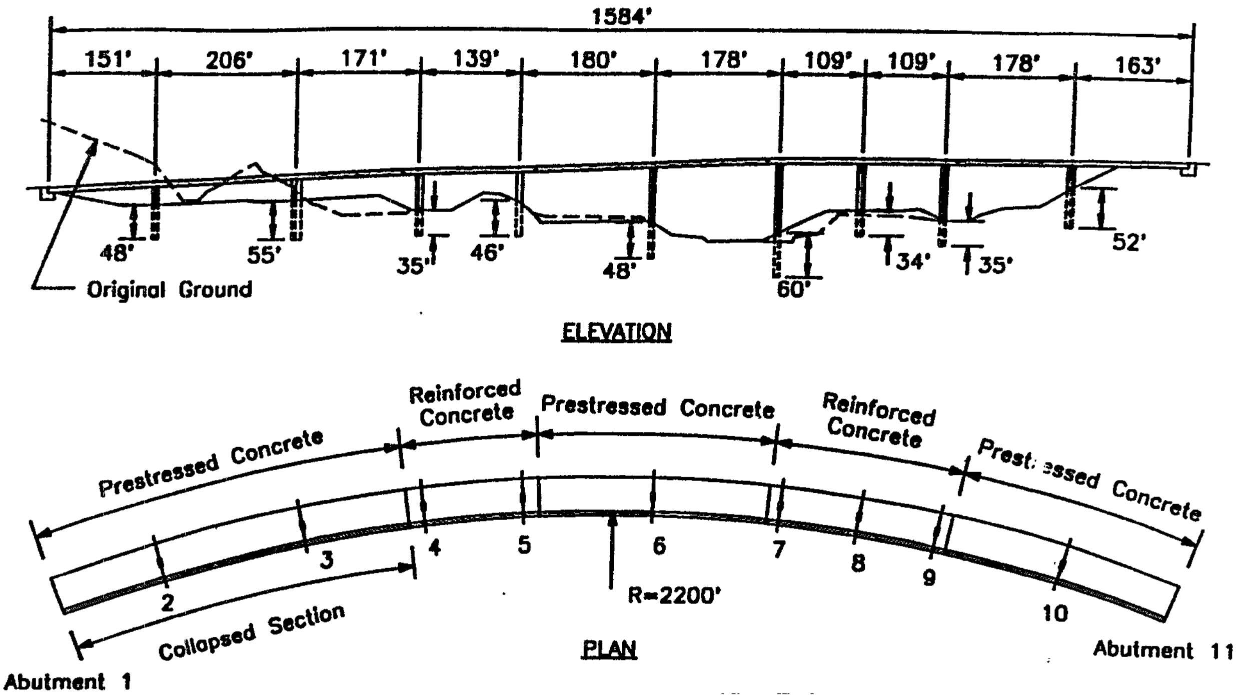
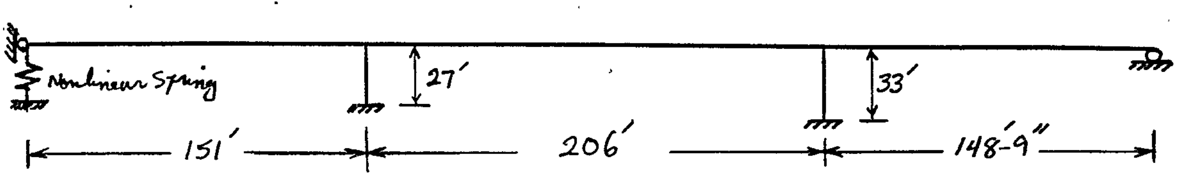


FIG. 1. General Elevation and Plan (from Priestley *et al.*, 1994)



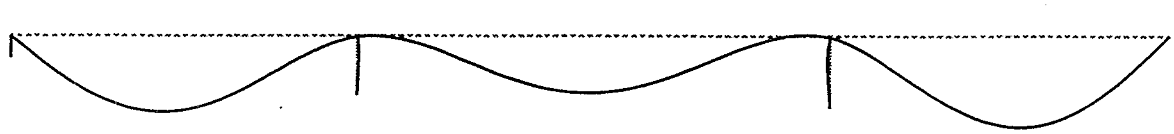
Superstructure:

I = 670 ft^4
A = 89.6 ft^2
E = 4,400 ksi
γ = 0.175 kips/ft^3

Columns:

I = 64 ft^4
A = 48 ft^2
E = 4,400 ksi

FIG. 2. Analytical Model Used for Time History Analysis



f = 2.960 Hz, α = 1.0

FIG. 3. Dominant Mode Shape in the Downward Direction

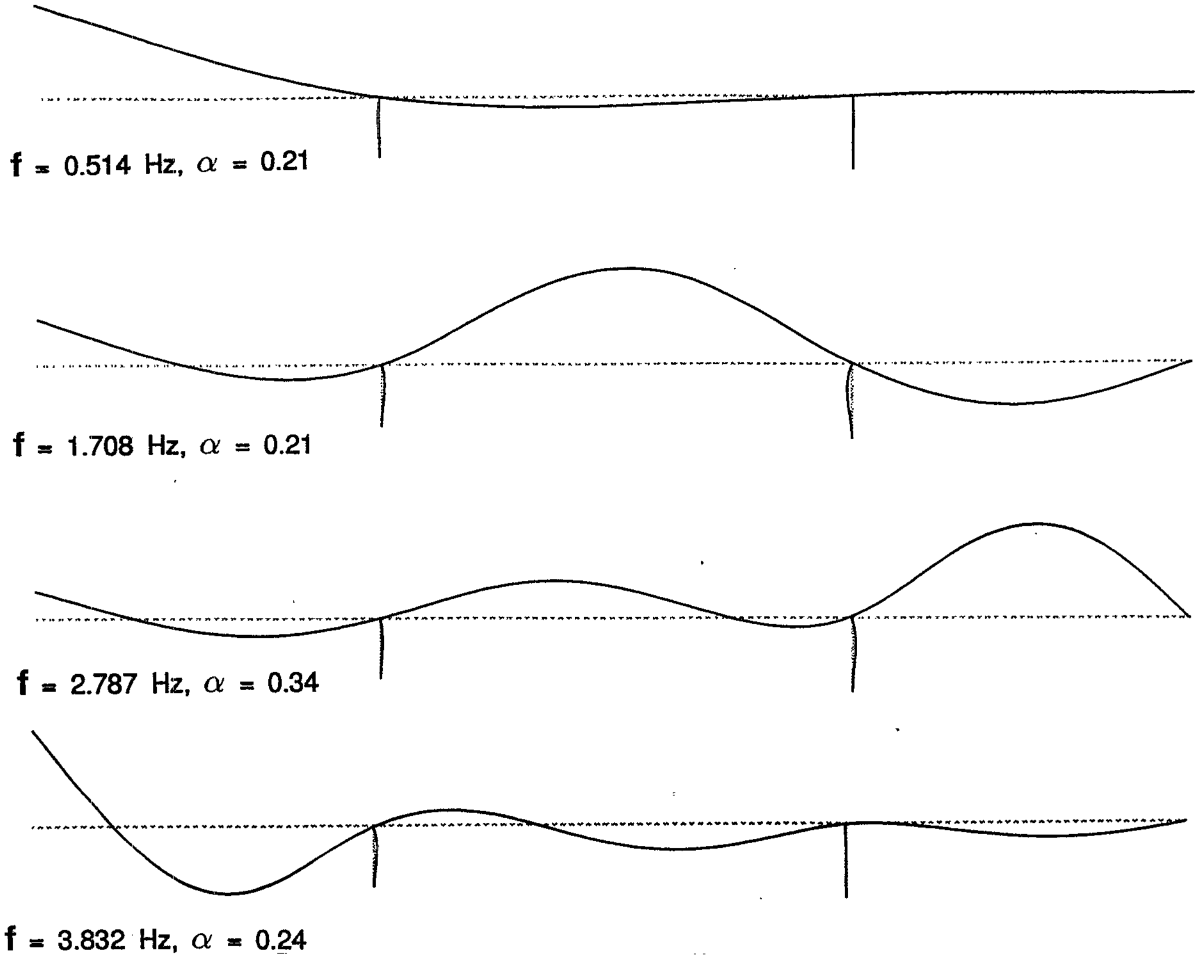


FIG. 4. Dominant Mode Shapes in the Upward Direction

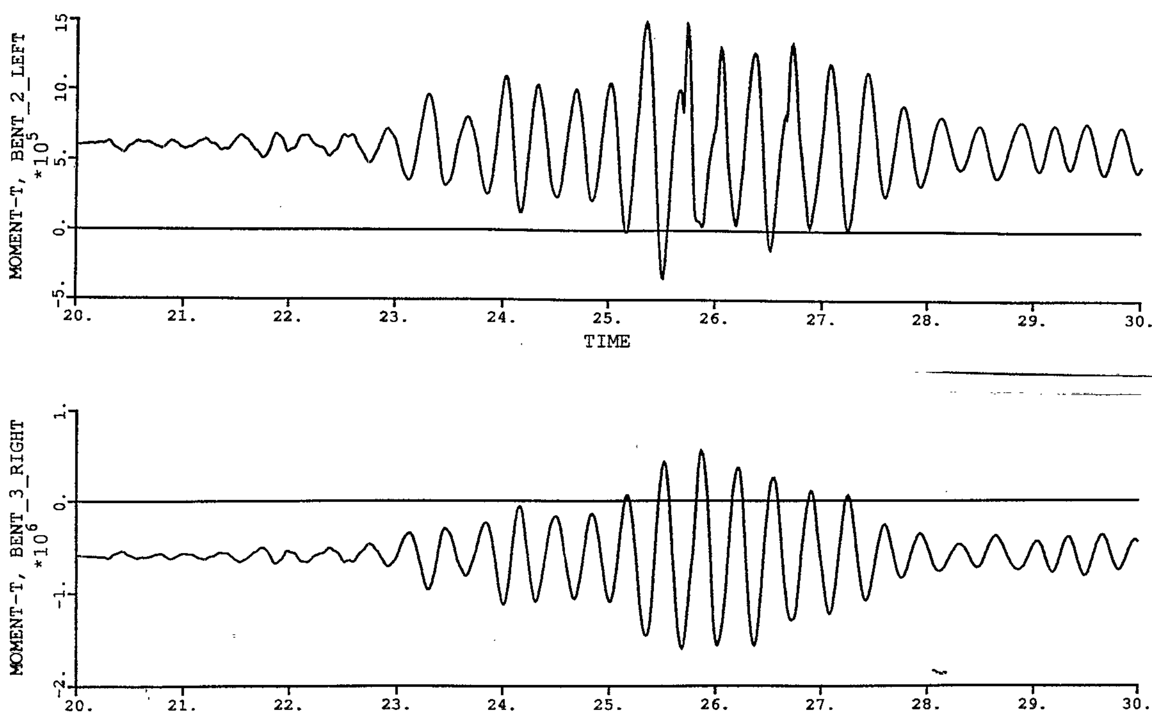


FIG. 5. Time Histories of Moments at the Right End of Left Span and Left End of Right Span