DYNAMIC ANALYSIS OF AN UNSYMMETRICAL HIGH RISE BUILDING

by

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SUMMARY

A complete time history response dynamic analysis is carried out to establish the design seismic loading for the Vancouver Square building. The building has an elevated observation deck and restaurant offset from the centre of the structure. The paper describes the steps and considerations involved such as the dynamic modelling of the structure, the choice of input ground records and the interpretation of the computed results. Whenever possible, the calculated values are compared with the National Building Code of Canada (1975) requirements to provide a proper perspective of the various approaches in establishing design loads for buildings.

INTRODUCTION

In the design of high rise structures in seismicly active areas, the effects of earthquakes become a predominant consideration in the design of the building. Three alternatives are available to estimate the design load due to earthquake; namely, an equivalent static loading approach as suggested by the National Building Code of Canada (NBC) (5), a dynamic analysis based on the response spectrum technique (2), and a complete dynamic response analysis to obtain the time history response of the proposed structure. Depending on the degree of complexity of the structure one may chose the most appropriate approach.

The present paper describes a study on the seismic analysis of an unsymmetrical high rise structure in the Vancouver area. The structure is a reinforced concrete frame symmetrical in one direction but asymmetrical in the other direction. Due to the seismic activity of the site and the unusual plan layout, it is felt necessary to carry out a complete dynamic time history analysis in order to obtain the distribution of interstorey shear forces and torsional moments for the proposed structure. For the earthquake loading in the symmetrical direction, a comparison is made between the equivalent static loading approach, the dynamic response spectrum approach and the complete dynamic response time history calculation. Such a comparison is useful to assess the appropriatness of the ground motion records used in the dynamic time history analysis. In the asymmetrical direction, the complete dynamic analysis gives the applied torques along the height of the structure, in additon to the interstorey shears and overturning moments.

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The purpose of this paper is to provide an example on the use of complete time-history analysis to establish the design earthquake loads for an unsymmetrical building with an unusual structural configuration. It shows the steps and the special considerations involved in the process wherever possible. Comparisons are made to the NBC 1975 to put the results obtained into the proper perspective.

DESCRIPTION OF BUILDING

The building described is the main building of the Vancouver Square complex in the downtown area, which is shown in a perspective in Figure 1 and in elevations in Figure 2.

It consists of a 5 storey substructure of which 3 are used for parking, shipping and receiving; a 5 storey section above grade which is linked with an existing 7 storey building to form together a large Department Store, a 21 storey office building above the store and an elevated observation tower with a revolving restaurant.

The main tower having a 116' by 116' plan size is cut loose from the surrounding construction above Hastings Street by expansion joints. The building is generally of reinforced concrete construction with the office tower floors constructed in lightweight concrete.

The perimeter basement walls and a system of interior shearwalls create a very stiff base from which the tower cantilevers to a height of 455 feet.

The department store floor and typical office floor framing are shown on Figures 3 and 4. The shaft supporting the observation tower is shown on Figure 5 and is offset 42' from the centre of the office tower below.

Columns are 28' on centres except along the perimeter of the office section of the tower where intermediate columns are added at 14' $^{o}/o$ to create a strong perimeter framing to better resist the torsional moments caused by the earthquake due to the offset observation tower.

The centre of resistance of the tower frame has been estimated to be about 8 feet to the south of the centre of the square building, which has been assumed to be also the centre of the mass of the building.

The observation and restaurant floors are framed in steel supporting a lightweight concrete floor deck.

DYNAMIC ANALYSIS

Two important phases in a complete dynamic analysis are the realistic choice of the input ground motion record and the dynamic modelling of the structure.

a) Choice of Input Ground Motion Record.

Geological studies in the southern British Columbia area indicate that most of the faults in the area are running in a north-west to south-east direction. The main ones are running from the Seattle area north-west past the Vancouver Island into the Pacific. Strong ground shaking in Vancouver is likely caused by slippage along these faults. The present site considered is thus located at a distance of 50-60 miles from the potential fault line. There is no measured ground acceleration record of significant magnitude in the Vancouver area that can be used as input to the dynamic analysis. Therefore, artifically generated earthquake records must be used, such as those designated as B-1 and B-2, generated by Jennings, Housner and Tsai(3). These records have a peak acceleration of about 31% g and a duration of 50 seconds. They model the ground motion expected at locations at a moderate distance from the epicentre in a magnitude 7 earthquake. They are comparable in intensity to the 1940 El Centro N-S record. For the present dynamic analysis, these records are scaled down to a peak acceleration of 10% g. This value of peak acceleration is consistent with the value suggested by EMR Earthquake Probability Analysis (7) and also by Khanna and Gadsby (4).

The response spectra of these two records is shown in Figure 6. Superimposed on the figure is the design elastic spectrum as suggested by the National Building Code of Canada (6) reduced to 10% g peak ground acceleration. It can be seen that the B-1 record gives a stronger shaking than the B-2 record, although both have the same peak acceleration value. The spectrum suggested by the NBC 1975 is approximately the average of the spectra of B-1 and B-2 records.

b) Dynamic Modelling

The structure is divided into seven regions. The masses of each region are lumped at the centre of the region. The stiffness of the structure is modelled by flexural, shear and torsional springs between the masses.

In the N-S direction, the structure is symmetrical and a planar analysis is sufficient to determine the response. The discrete dynamic model is shown on Figure 7. The arrangement of the shear spring and flexural springs between the masses is to take into account the web drift and chord drift of each region as shown in Figure 8. Such a dynamic model is a refinement to the usual shear beam or flexural beam approximation of multistorey structures. Since the axial deformation of the columns has a significant effect on the lateral stiffness of high rise structures, it is necessary to take chord drift into account in the modelling.

In the E-W direction, due to the offset of the elevator shaft and the top appendage, a three-dimensional analysis becomes necessary to take into account the coupled flexural torsional response of the system. The discrete model shown in Figure 8 is expanded by inclusion of linear torsional springs between the masses and also by allowing that the plane containing the mass centre of each region is not necessarily the same plane containing the center of resistance for that region. Figure 9 shows the dynamic model for excitation in the E-W direction.

It is assumed that the lines of mass centres and centre of resistance coincide approximately in the first six zones while the lines of mass centres and centres of resistance in zone 7 are offset a distance of 34 feet from the zones below. In this zone, the stiffness is provided by the elevator and stair shafts. Therefore, it is modelled as a beam with the same moment of inertia flexurally, and as a closed equivalent square section for torsional stiffness calculations.

RESULTS AND DISCUSSIONS

It is useful to discuss the results of the analysis separately for the N-S direction and the E-W direction. Since the N-S excitation causes planar lateral response only, a comparison can be made for the various approaches in seismic analysis as suggested by code. Excitation in the E-W direction causes torsional response in additional to flexural response.

(i) North-South Direction Responses (planar response).

The first six natural periods are shown in Table I. Although only the periods associated with lateral vibrations are of interest in the N-S direction response calculation, the periods associated with torsional vibration are also presented. Such a presentation will be helpful to appreciate the coupled lateral torsional vibration problem to be discussed later on.

The first three lateral mode shapes are shown on Figure 10. If the chord drift is neglected so that the building is treated as a shear building, the stiffness of the structure will be over-estimated by 20%, resulting in a first natural period of 3.48 seconds, instead of 3.74 second.

The torsional mode shapes are shown in Figure 11. The mode shapes are normalized so that the maximum co-ordinate is unity. The torsional mode shapes differ from the flexural mode shapes. The node points at the higher torsional modes are closer to the base of the structure as compared to those in flexural modes.

Once the natural periods and mode shapes are known, the response can be obtained using the response spectrum technique as suggested by NBC 1975 (6). Assuming a damping value of 5% critical in the first three modes of lateral vibration and using the elastic design spectrum suggested by the code, the shear force acting on the structure is obtained. Shown on Figure 12 are the shear force envelopes based on the first mode response, and based on the root sum square (RSS) of the first three lateral mode responses. As a comparison, the shear force diagram according to equivalent static load approach is also shown. The computation of the equivalent static forces is based on the following parameters: A = 10%g, T = 3.74 second (fundamental period based on dynamic analysis), K = 0.7, I = F = 1.0.

Comparison between the single mode response envelope, and RSS three mode response indicates that major differences exist at the top and bottom of the building. In terms of percentage difference, the shear force calculated based on the 1st mode response is 50% below the shear based on the RSS of three modes. The importance of higher modal constributions for this structure is evident.

Comparison between the shear force envelope based on the equivalent static force approach and those based on the response spectrum technique indicates that the former is similar to the fundamental mode response, but smaller than the RSS 3 mode response.

However, it should be noted that the modal response envelopes are calculated using the elastic design spectrum. If a modest value of ductility ratio $\mu = 2$ is assumed, one can reduce the RSS 3 mode response by a half, resulting in a smaller interstorey envelope than that based on the equivalent static force approach. A ductility ratio of two may be considered a minimum ductility value that can be expected of a building designed based on a structural K factor of 0.7. Therefore, the equivalent static force approach appears to give a conservative design value in this case.

In calculating the time-history reponse, a step-by-step integration procedure is used, assuming the acceleration between each time interval being linear (1). Damping values of 5% critical are used for the first and second flexural modes.

The shear response envelope due to the B-1 and B-2 earthquake records are shown on Figure 13. To check on the sensitivity of the response to the stiffness values of the structure, the stiffness of the structure is increased and decreased by 20%. The response envelopes for these cases are shown on Figure 14. It can be seen that the difference is of the order of 5-10%, with the stiffer structure in general giving a higher value of interstorey shear.

Shown in Figure 15 are the shear envelopes due to the response spectrum technique and the average shear response of the structure due to records B-1 and B-2. Shown in a solid line is the shear envelope actually used in the design. The choice of the design shear envelope is a matter of judgement in which the following factors must be considered:

(i) the uncertainty of the peak acceleration values (from 6%g to 20%g).
(ii) the results obtained through the response spectrum study.
(iii) the results of the dynamic analysis using the B-1 and B-2 records.
(iv) the structural configuration of the building.

It is seen that the recommended design shear follows fairly closely the curve obtained by the response spectrum technique and that it averages 75-80% of the shear based on dynamic analysis.

In view of the past experience on the performance of appendages to buildings during earthquake, and since the supporting structure for the observation and restaurant floors are essentially a series of concrete shafts, the "neck" portion of the tower is a vulnerable part of the structure. Therefore, the design shear value for this "neck" is taken to be larger than the calculations indicates.

It should be pointed out that the response spectrum analysis and dynamic analysis are based on an elastic behaviour of the building frame. If the structural frame was designed for the recommended shear values and a ductility of at least 2 it should easily resist earthquake motions of 10% g peak acceleration and should be able to withstand motions of 20%g peak accelerations without collapse.

From the design shear envelope one can determine a set of quasi-static lateral loads which can then be used to determine the internal forces in the concrete frame by a computer frame analysis.

(ii) East-West Direction Response.

When the structure is excited in this direction coupled flexural-torsional vibrations take place due to the eccentric arrangement of the elevated observation tower. The natural modes consist of both lateral and rotational displacements. For some modes, the mode deformation consists mainly of lateral displacements with a minor component of rotation of the building. These modes are termed flexural predominant modes. On the other hand, some modes consist mainly of rotation and these modes are called torsion predominant modes. The modes are numbered in an ascending order according to their periods. The first six coupled mode periods are shown in Table I. An examination of the mode shapes indicates that the first, third and fifth modes are torsion predominant modes while the second, fourth and the sixth modes are flexure predominant modes.

It can be seen that the periods of the first torsion predominant mode and the first flexure predominant mode are modifications of the first torsional mode period and first flexural mode period in the N-S direction. The effect of coupling is to separate the periods of the corresponding uncoupled modes. Similar observations can be made to the third and fourth, and fifth and sixth modes of the coupled vibrations.

As for the north-south direction, the records B-1 and B-2 are used as inputs to obtain the dynamic time-history response of the structure. 5% damping is used for the first two flexural predominant modes. It is found that the shear response envelopes are similar to the corresponding envelopes in the N-S direction response calculation. Therefore, the design shear envelope used in the N-S direction response calculation is also suitable for the E-W direction.

In addition to the shear forces, torques are also induced in the structure. The torque envelopes are shown in Figure 16. It appears that the shape of the envelope is sensitive to the record used. The building is subjected to almost a constant torque using record B-2 while the torque envelope for record B-1 increases towards the base of the structure. The nominal stiffness value of the structure is increased and decreased by 20% to check the sensitivity of the torque envelope. The resulting response envelopes are shown on Figure 17. On account of the complex nature of coupled vibrations, it is difficult to observe any orderly trend among the three envelopes.

The sensitive nature of the shape of the torque envelope, both with respect to the record used and the variation of structural stiffnesses strongly suggests to view the results of the analysis as basically providing only "order of magnitude" information. The average of the two response envelopes shown on Figure 16 will provide reasonable guidance in this respect. To incorporate this information in design, one can consider that at any level, the applied torque is resisted by two pairs of couples provided by the two pairs of perimeter frames. Although the internal frames will also participate in resisting the applied torque, it is prudent to ignore this participation in estimating the added shear forces to the perimeter framing. This additional shear in the perimeter frame should combine with the lateral shear forces in such a way that any possible benefit derived from the induced torsional moment is ignored.

It is of interest to interpretate the NBC code provision with respect to torsion in buildings. It is assumed that there is no eccentricity in the main portion of the building. The eccentricity arises because of the offset of the observation tower. In the dynamic model, we consider the top mass offset 34' from the mass centre line of the lower portion. Therefore, the torque applied to the whole structure is equal to the shear force at the base of the offset observation tower times the eccentricity due to the offset. Since the eccentricity is zero at levels below the offset observation tower, no additional torque will result. Therefore, if we use the NBC 1975 code provision, the torque envelope will be constant along the height of the building.

Let us consider the magnitude of this torque envelope. Using a K factor of 0.7 in the static equivalent force calculation, the shear at the base of the offset observation tower (or the top of the 21st office floor) is 508 kips. The calculated eccentricity being 34 feet, the design eccentricity $e_x = (1.5)\ (34) + 0.05\ (D_n)$. With the width of the building of 116 feet, the design eccentricity becomes 56.8 feet. Since the eccentricity exceeds a quarter the width of the building, the NBC code provision suggests doubling the effects of torsion, resulting in an eccentricity of 114 feet. Using a shear force of 508 kips, the estimated torsional moment is therefore 58,000 ft. kip. The torque envelope according to this calculation is also shown on Figure 16. A comparison between the torque envelope suggested by the code and those by dynamic analysis shows that the code gives a conservative estimate at the top part of the building but underestimates the torque at the base of the structure.

CONCLUSION

This paper describes the dynamic procedure carried out to determine the seismic loading of the Vancouver Square Tower. Due to the unsymmetrical nature of the building, a complete dynamic time-history analysis is performed to determine the interstorey shear and torsional moments acting on the structure. Whenever possible, the results based on the dynamic analysis are compared with the National Building Code of Canada 1975 provisions. Although the results presented are related to a specific building, the approach taken and the findings are believed to be useful for the design of similar buildings.

A comparison between the interstorey shear envelopes based on static equivalent load and those based on the elastic response spectrum technique shows that the static equivalent load approach gives similar values to the fundamental mode response envelope. If higher modal contributions are taken into account, the static equivalent load approach tends to underestimate the shear at the top and bottom of the structure. However, it should be remembered that the static equivalent load is calculated based on a K factor of 0.7. While there is no explicit interelationship provided in the code between the K factor and the ductility factor μ of the structure, it is reasonable to assume that if a structure qualifies for a K factor of 0.7, it has a ductility factor of at least two or three. If one assumes a ductility of two and uses the inelastic response spectrum as suggested by the code, one can reduce the shear envelope based on the response spectrum approach by a factor of one half, resulting in a smaller shear envelope than the envelope based on the quasi-static approach. From the designer's point of view, it would appear a sound design strategy to use the equivalent static approach to obtain initial proportioning of members and a response spectrum technique analysis to make refinements in the final design.

To obtain a dynamic time-history analysis, one of the problems is to use an appropriate ground record. It is shown that by comparing the response spectra of the records used and the design spectrum suggested by the code, it is possible to assess the appropriateness of the records. In the present case, the B-1 record provides a stronger excitation while the B-2 record gives weaker excitation than the "design ground disturbance", as measured by their spectra. An average of the responses from B-1 and B-2 would then give response similar to those excited by the "design ground disturbance".

The induced torsional moment envelope due to the unbalanced nature of an unsymmetrical building subjected to ground motions is very complex. It is sensitive to the actual record used and also sensitive to the possible variations in the structural stiffnesses. The response spectrum technique as applied to unsymmetrical structures is not well established. Research in this direction is needed. The code provision to estimate the torsional moment leads to an over conservative estimate at the top part but underestimates the torque at the lower part of the building. At the present state of the art, it is advisable to carry out a dynamic time-history analysis to estimate the torque envelope. Prudent judgement is required in distributing the influence of the torque to the various parts of the structure.

To design an unsymmetrical building against earthquake is a complex undertaking. There is no clear cut step-by-step procedure to obtain the design load. The analyst should try out different approaches to obtain a feel to the problem at hand, as many judgement factors are involved in choosing the final design values. The designers should be aware of the inherent uncertainty in the design load values and use them in an intelligent manner. Co-operation and understanding between the analyst and designer becomes an important ingredient in such an undertaking.

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	TOWER DESIGN:	M.S. Yolles & Parnters Ltd.
6.	GENERAL CONTRACTOR:	The Foundation Co. (Western) Ltd.

REFERENCES:

1. Clough, R.W. 1970, Earthquake Response of Structures, Chapter 12, Earthquake Eng., ed. by R.L. Wiegel.

2. Hudson, D.E. 1956, Response Spectrum Technique in Engineering Seismology, Proc. First World Conf. on Earthquake Engineering, Berkeley, California.

3. Jennings, P.C., G.W. Housner and N.C. Tsai, 1969. Simulated Earthquakes Motions for Design Purposes. Proc. 4th World Conf. Earthquake Eng., Santiago, Chile, A-1, pp. 145-160.

4. Khanna J. and J.W. Gadsby, 1972, Seismic Exposure in Greater Vancouver, EIC Journal, July issue, pp 37-42.

5. National Building Code of Canada 1975, subsection 4.1.7, Effects of Earthquakes.

6. National Building Code of Canada, 1975, Supplement No. 4, Commentary K. Dynamic Analysis for the Seismic Response of Buildings.

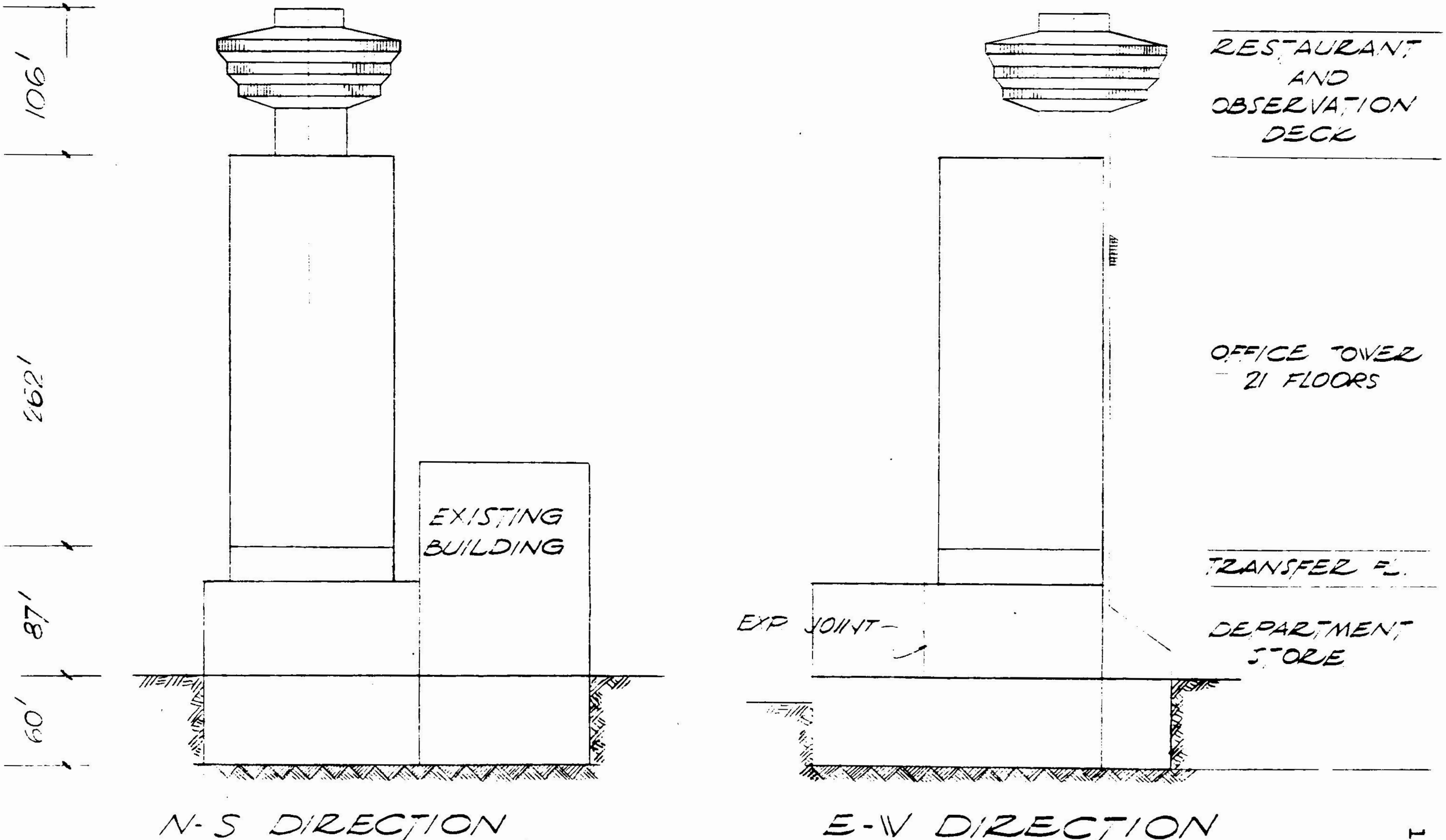
7. Seismic Evaluation, Div. of Seismology, Dept. of Energy, Mines and Resources, Ottawa, Canada.

LIST OF FIGURES:

1 Vancouver Square Complex
2 Elevations of Building
3 Typical Department Store Floor
4 Typical Office Floor
5 Observation Tower Shaft
6 Velocity Spectrum of Input Records
7 Mass Distribution for Excitation in N-S Direction
8 Discrete Dynamic Model
9 Mass Distribution for Excitation in E-W Direction
10 Lateral Mode Shapes
11 Torsional Mode Shapes
12 Interstorey Shear (N-S Direction)
13 Interstorey Shear based on Dynamic Analysis
14 Sensitivity of Shear Envelope
15 Summary of Interstorey Shears
16 Torque Envelope from Dynamic Analysis
17 Sensitivity of Torque Envelope

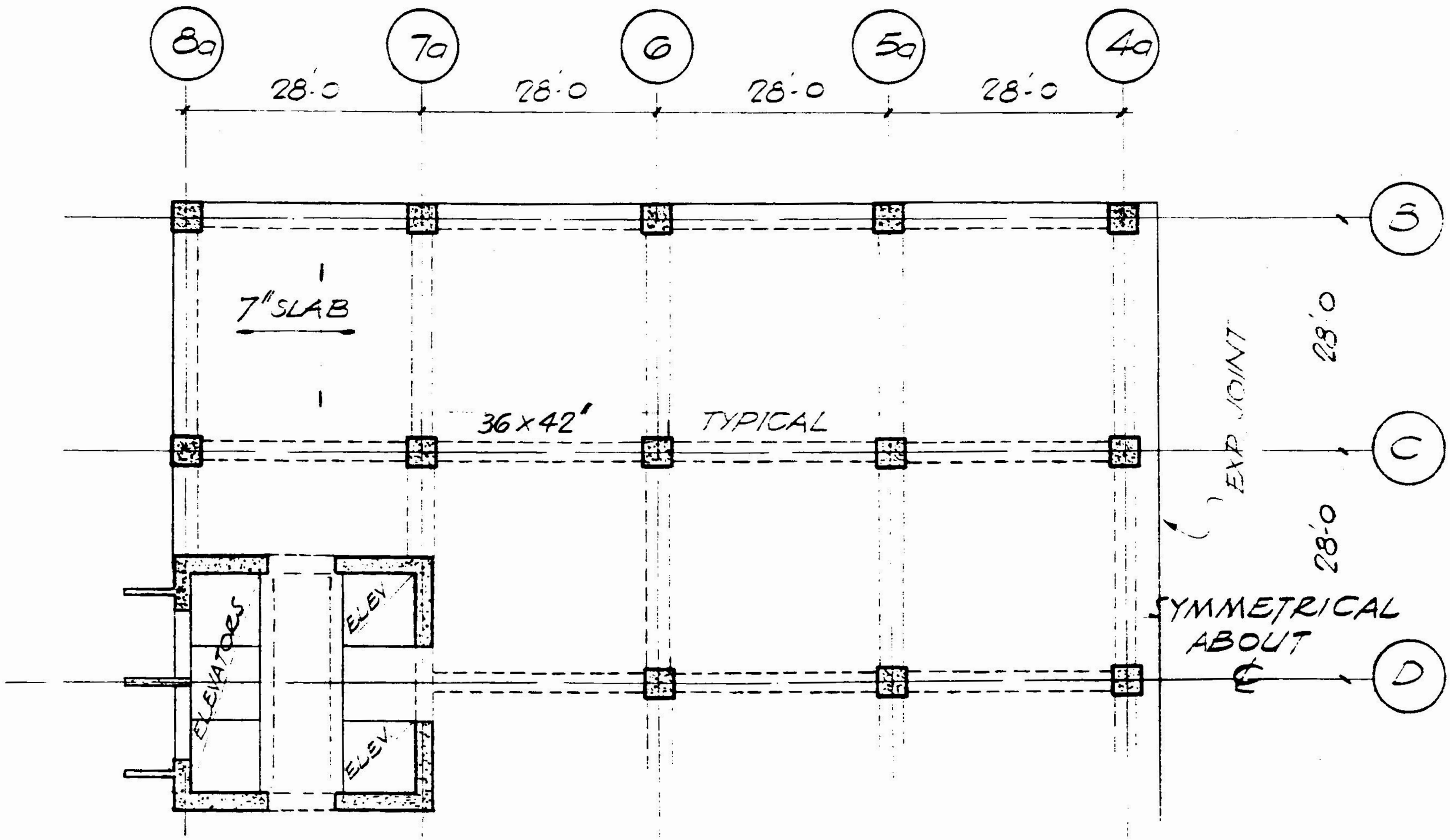
TABLE 1 Periods of Natural Modes.

THE VANCOUVER SQUARE COMPLEX
FIG. 1



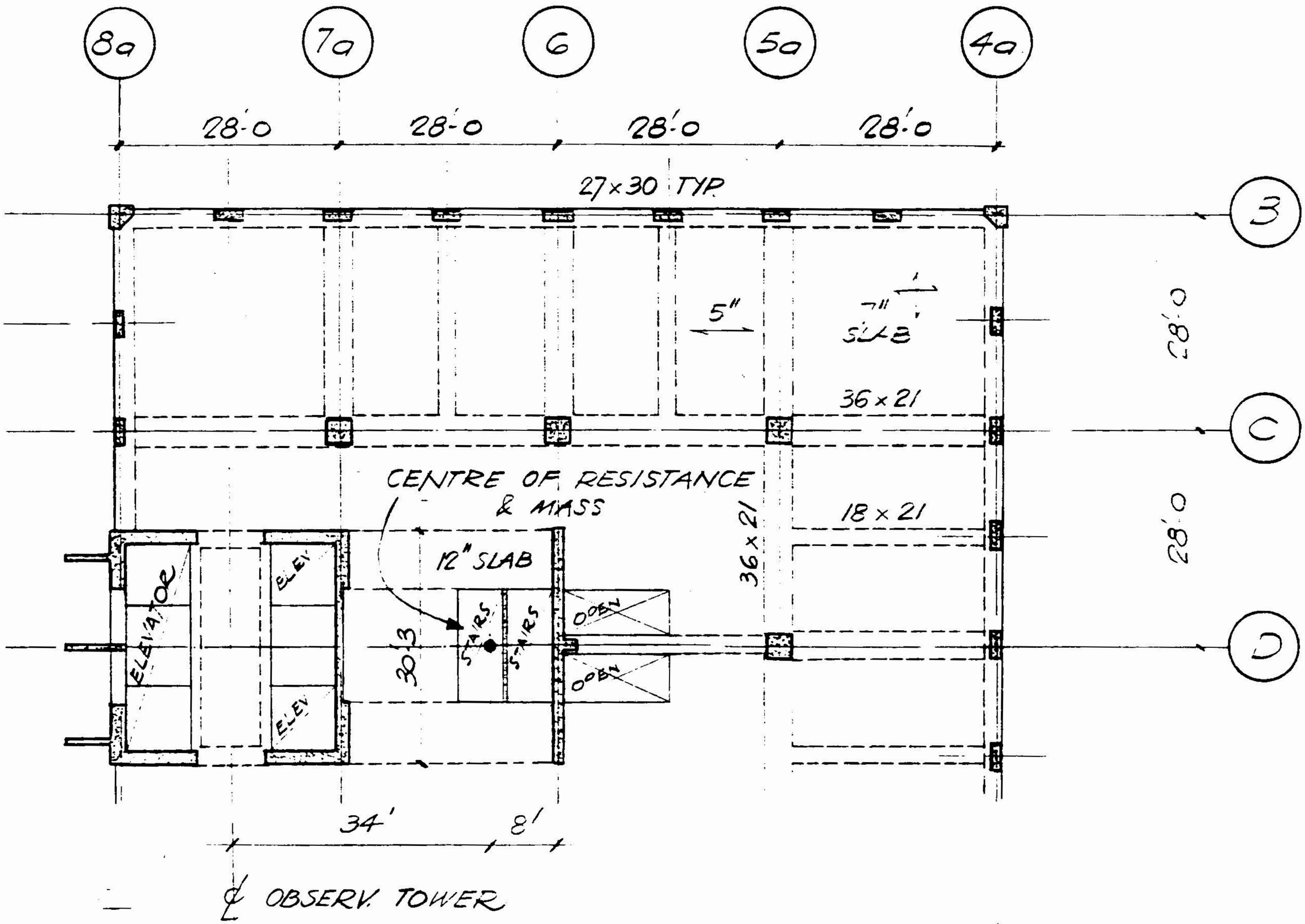
ELEVATIONS OF BUILDING

FIG. 2



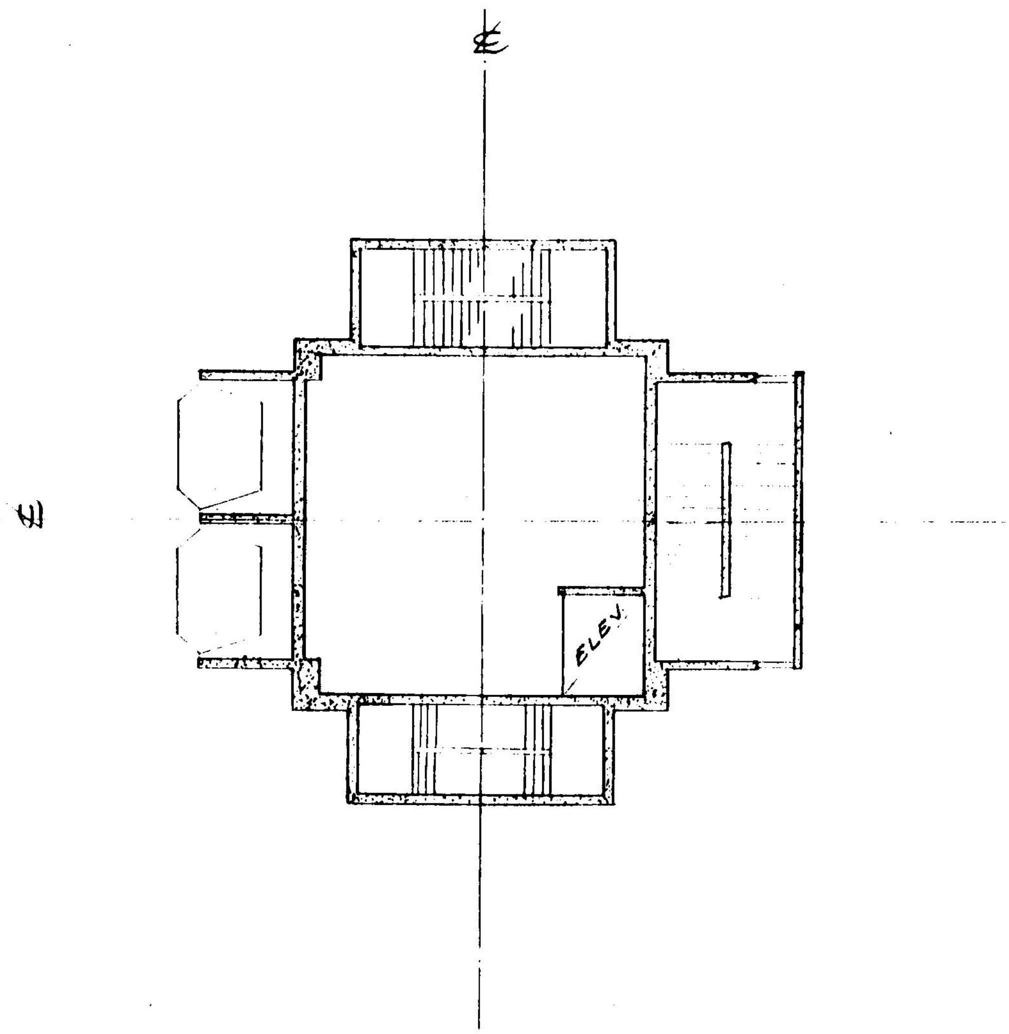
TYPICAL DEPARTMENT STORE FLOOR

FIG. 3



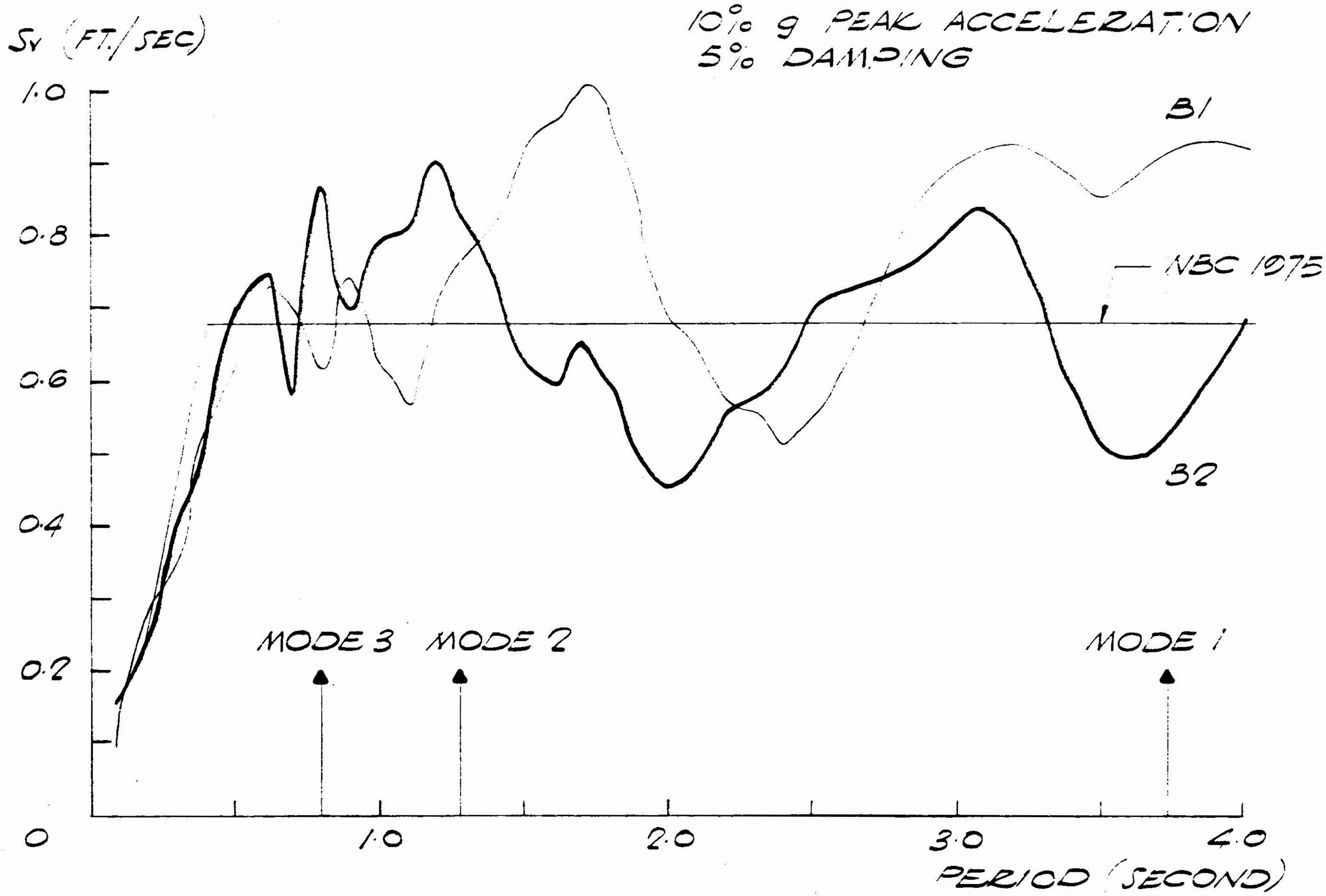
TYPICAL OFFICE FLOOR

FIG. 4



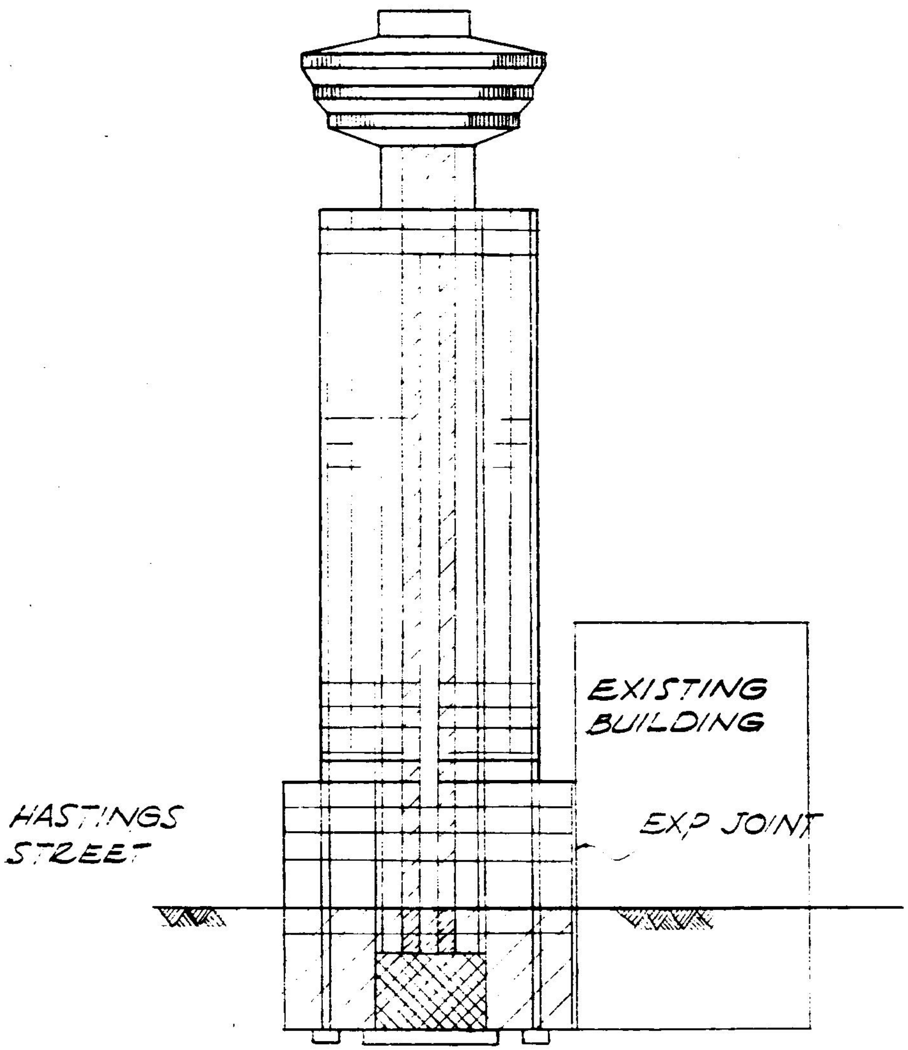
OBSERVATION TOWER SHAFT

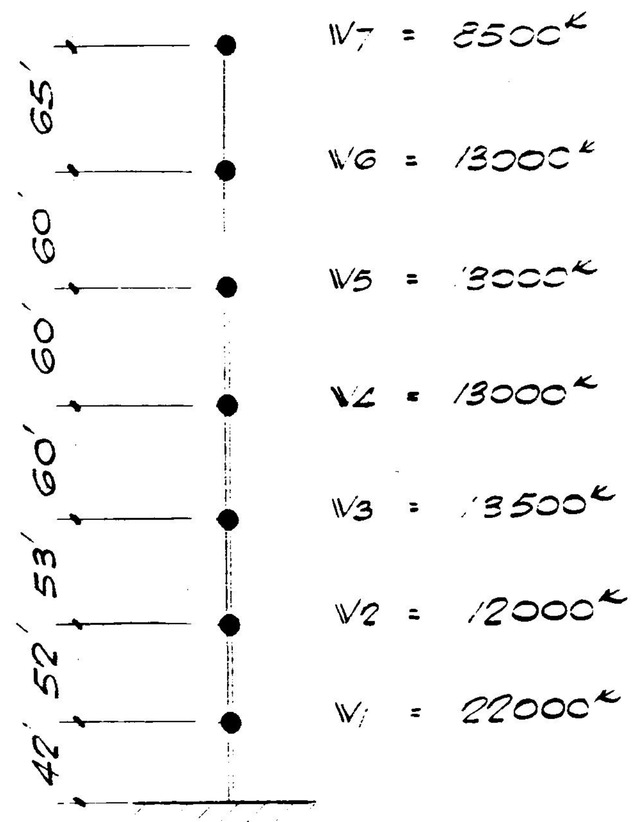
FIG. 5



VELOCITY SPECTRUM OF INPUT RECORDS

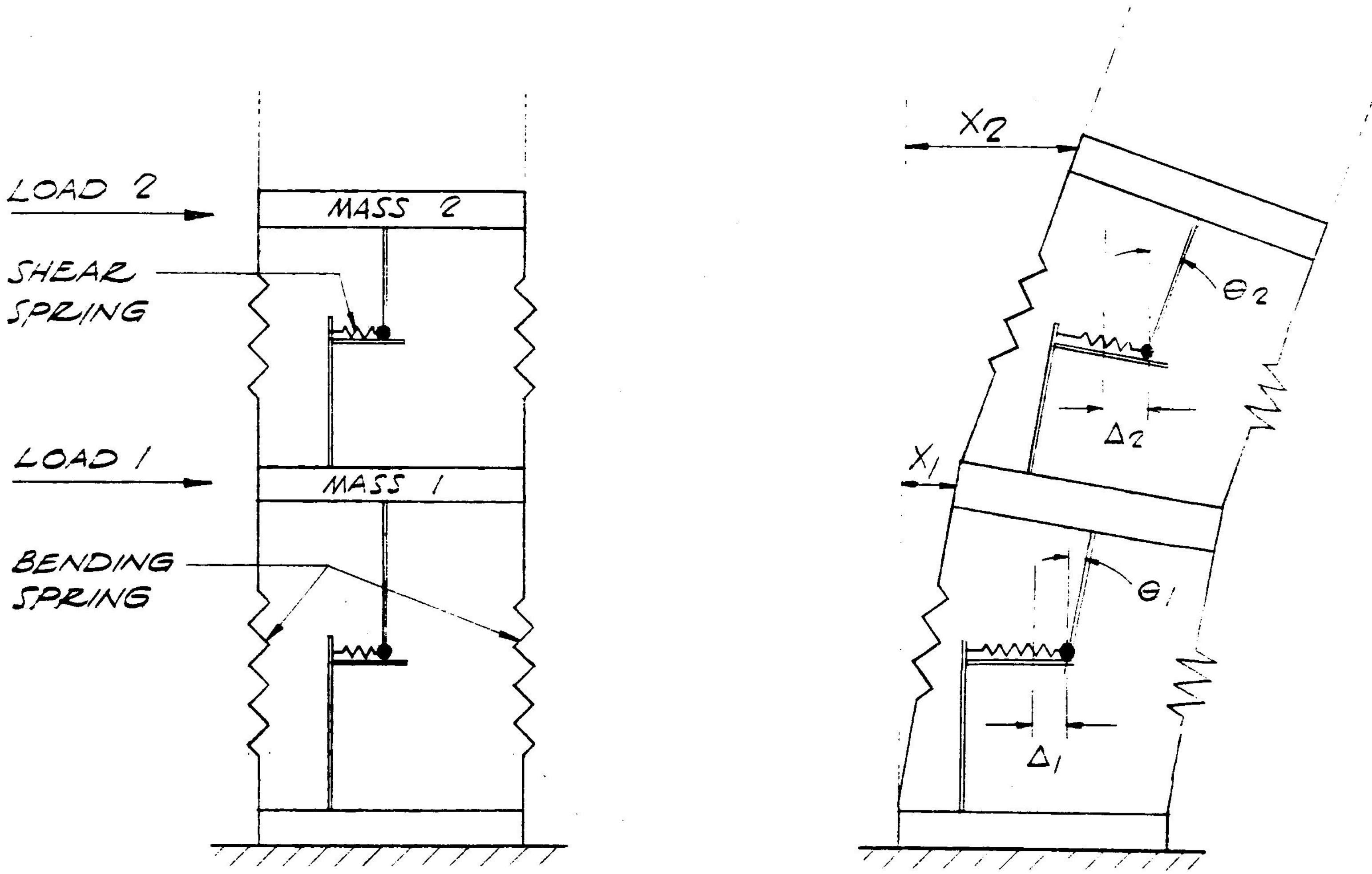
FIG. 6





MASS DISTRIBUTION FOR EXCITATION
IN NORTH - SOUTH DIRECTION

FIG. 7

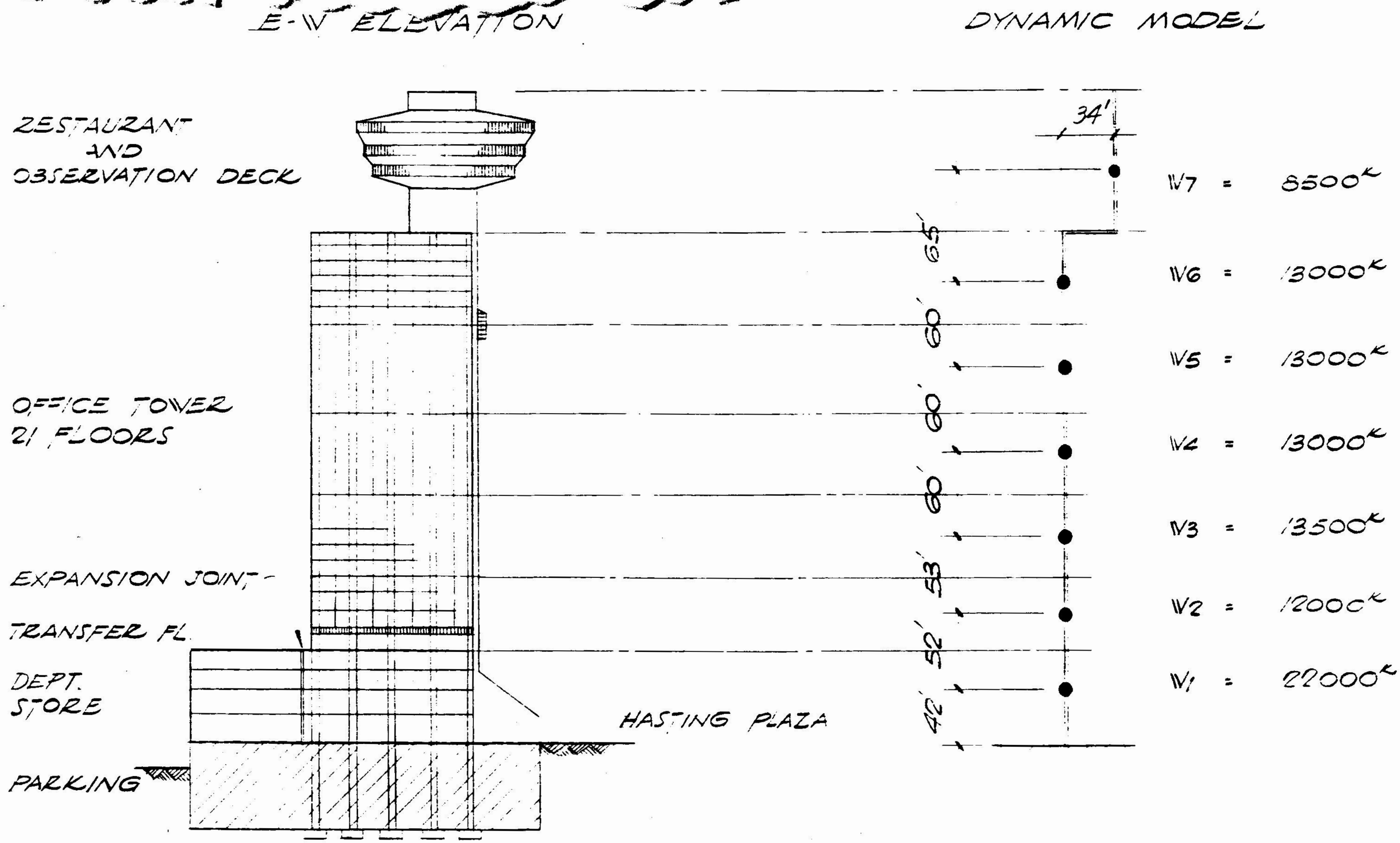


ORIGINAL POSITION DEFLECTED POSITION

X - TOTAL DEFLECTION
Δ - WEB DRIFT
Θ - ROTATION CAUSED BY CHORD DRIFT

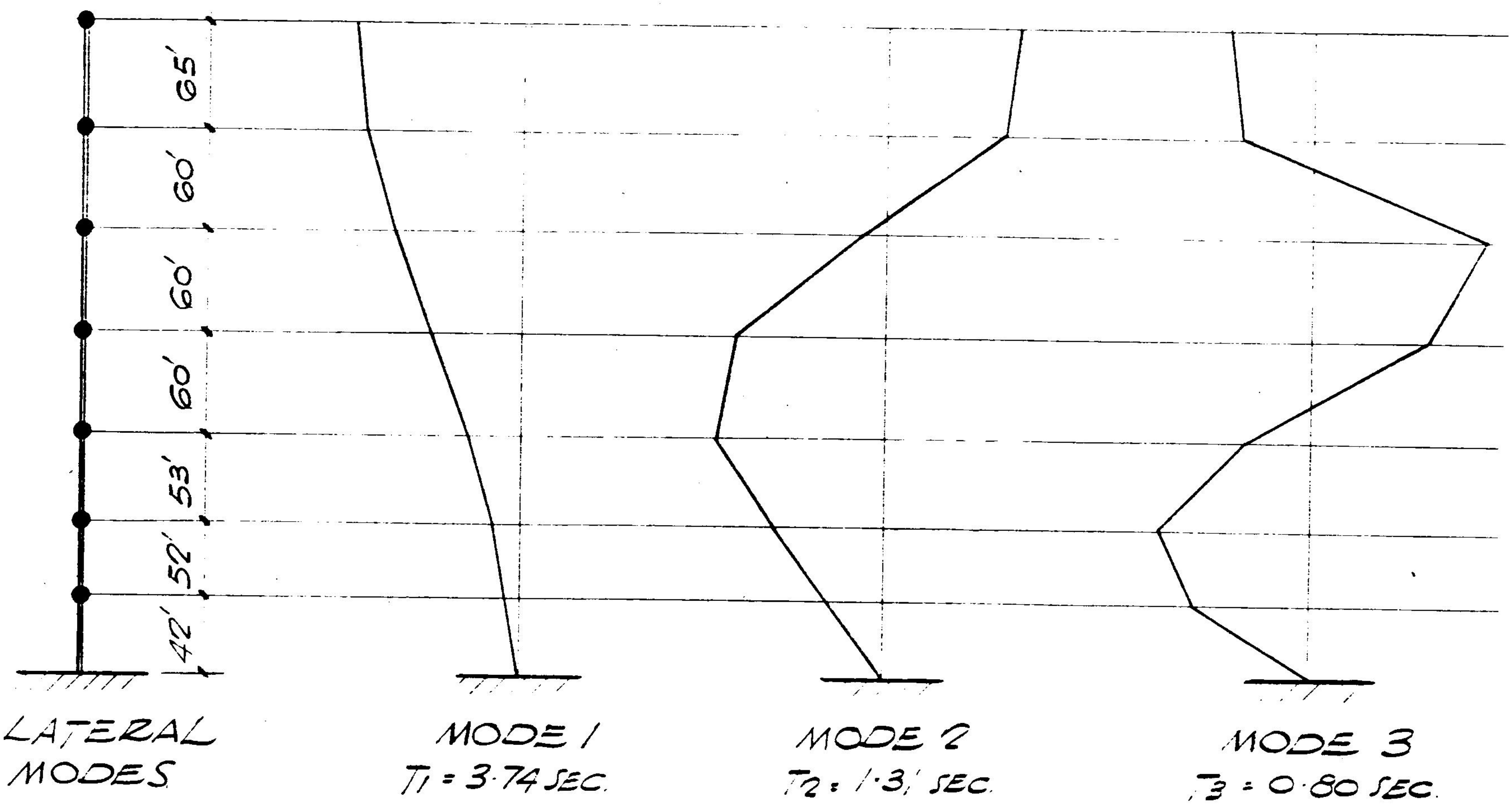
DISCRETE DYNAMIC MODEL

FIG. 8



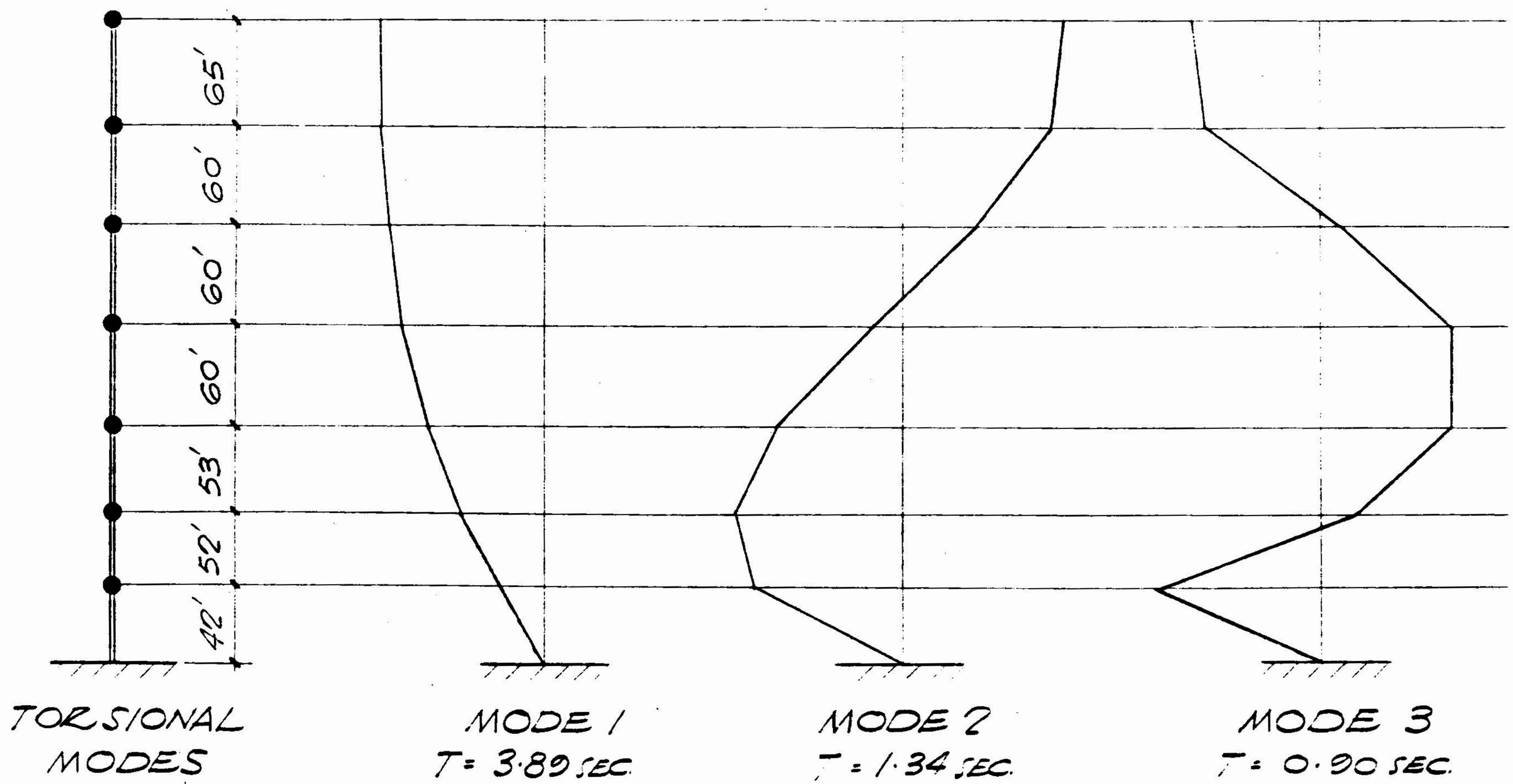
MASS DISTRIBUTION FOR EXCITATION IN EAST - WEST DIRECTION

FIG. 9



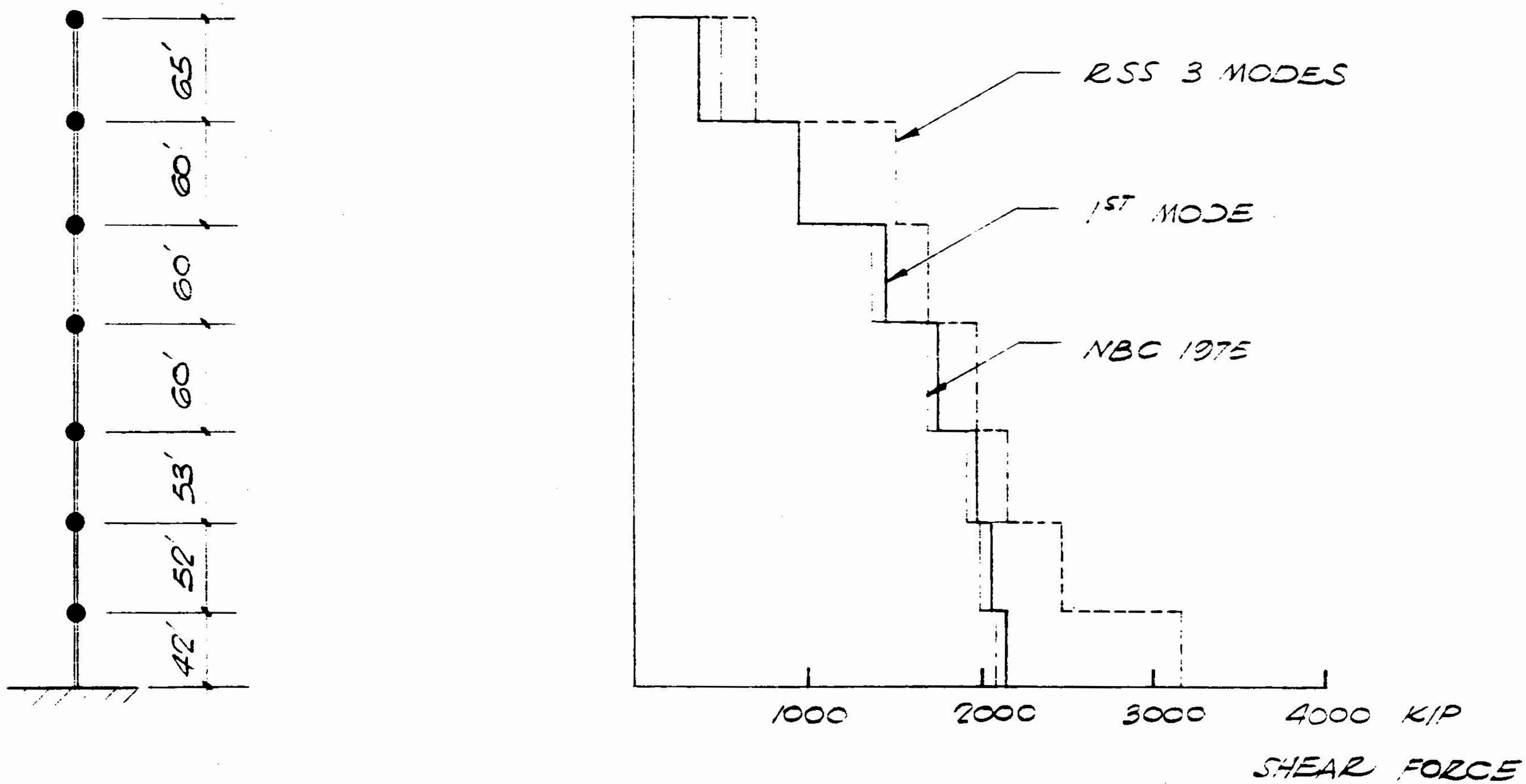
LATERAL MODE SHAPES

FIG. 10



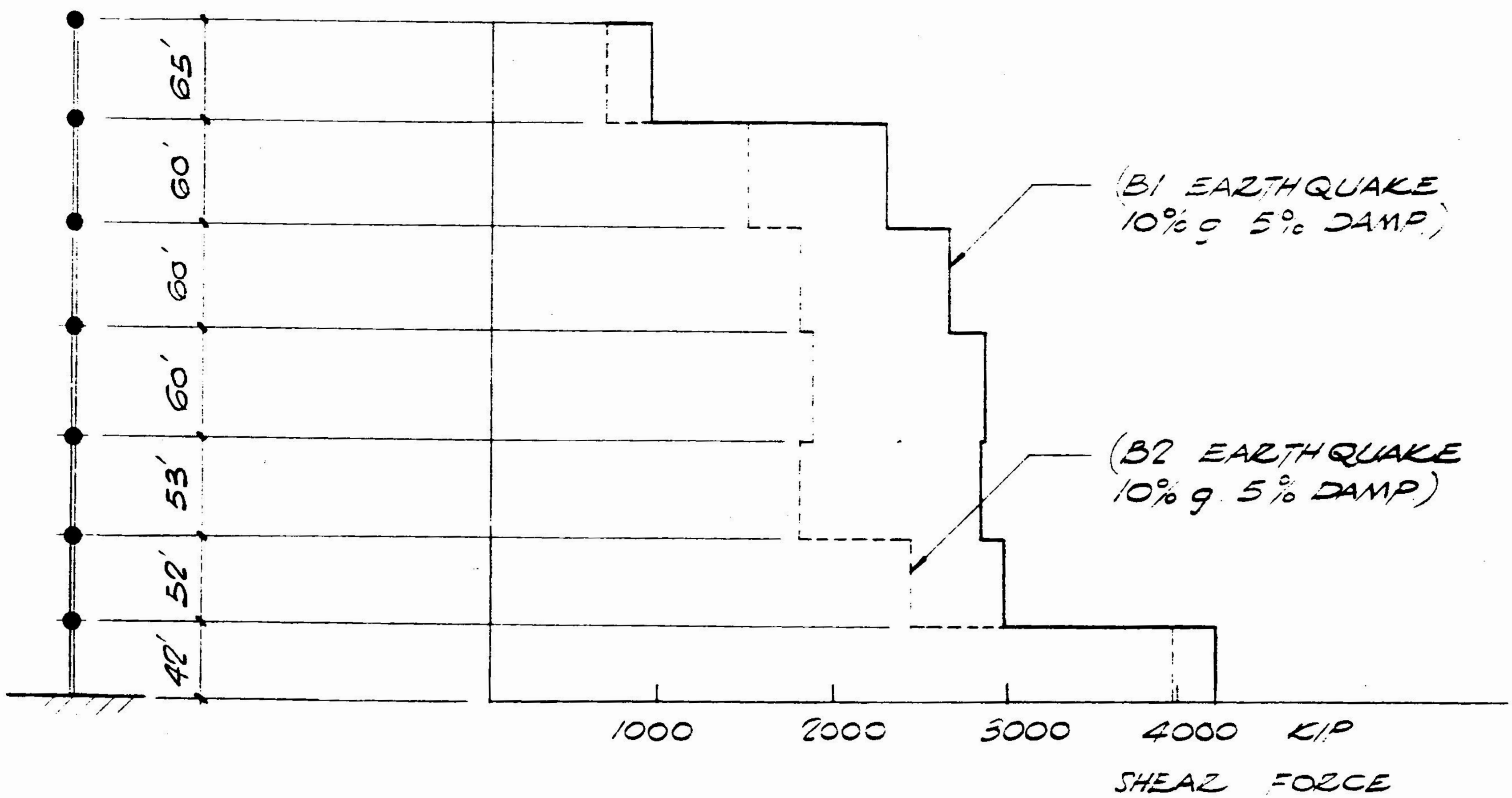
TORSIONAL MODE SHAPES

FIG. 11



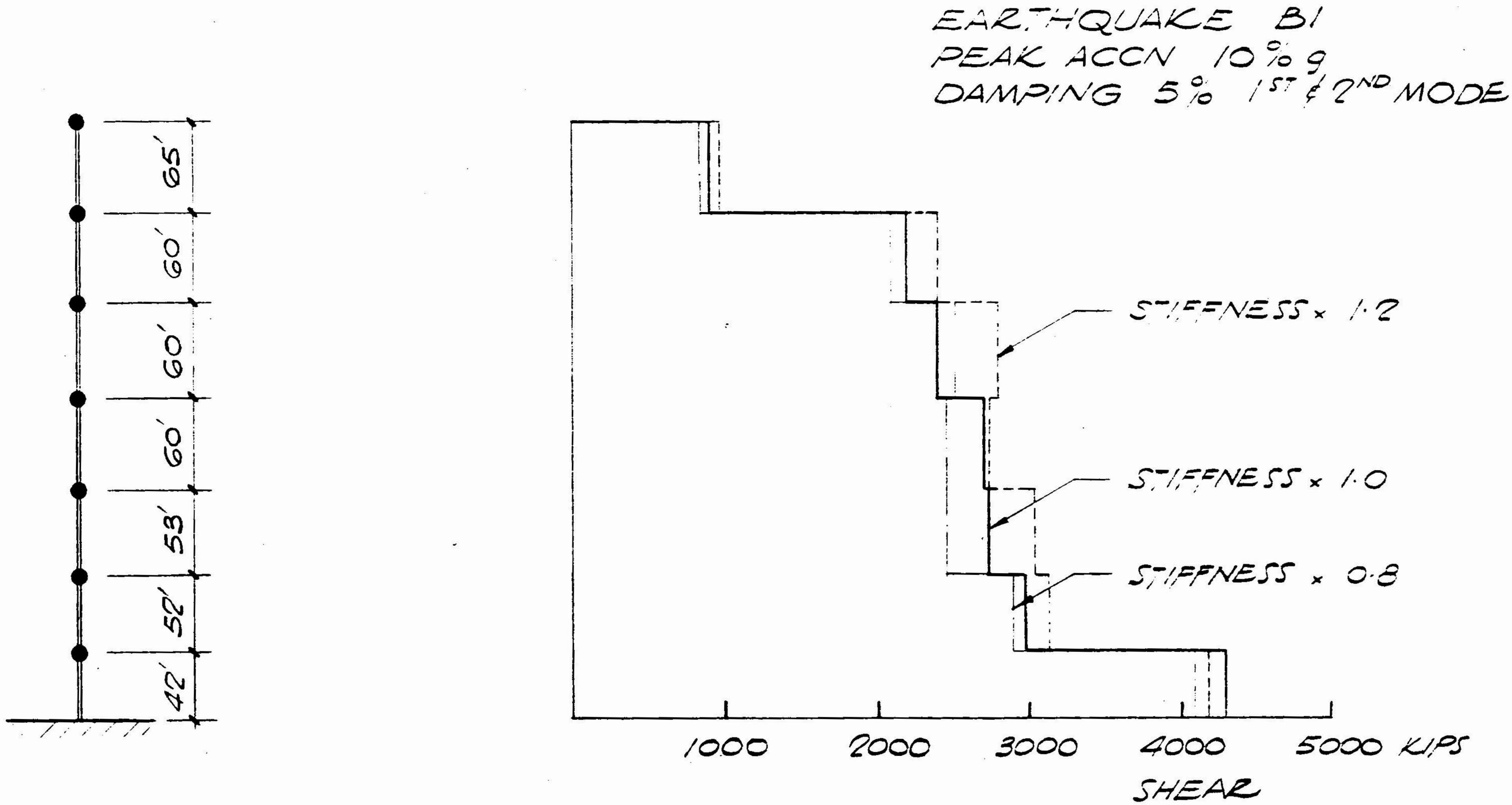
INTERSTOREY SHEAR (N-S DIRECTION)

FIG. 12



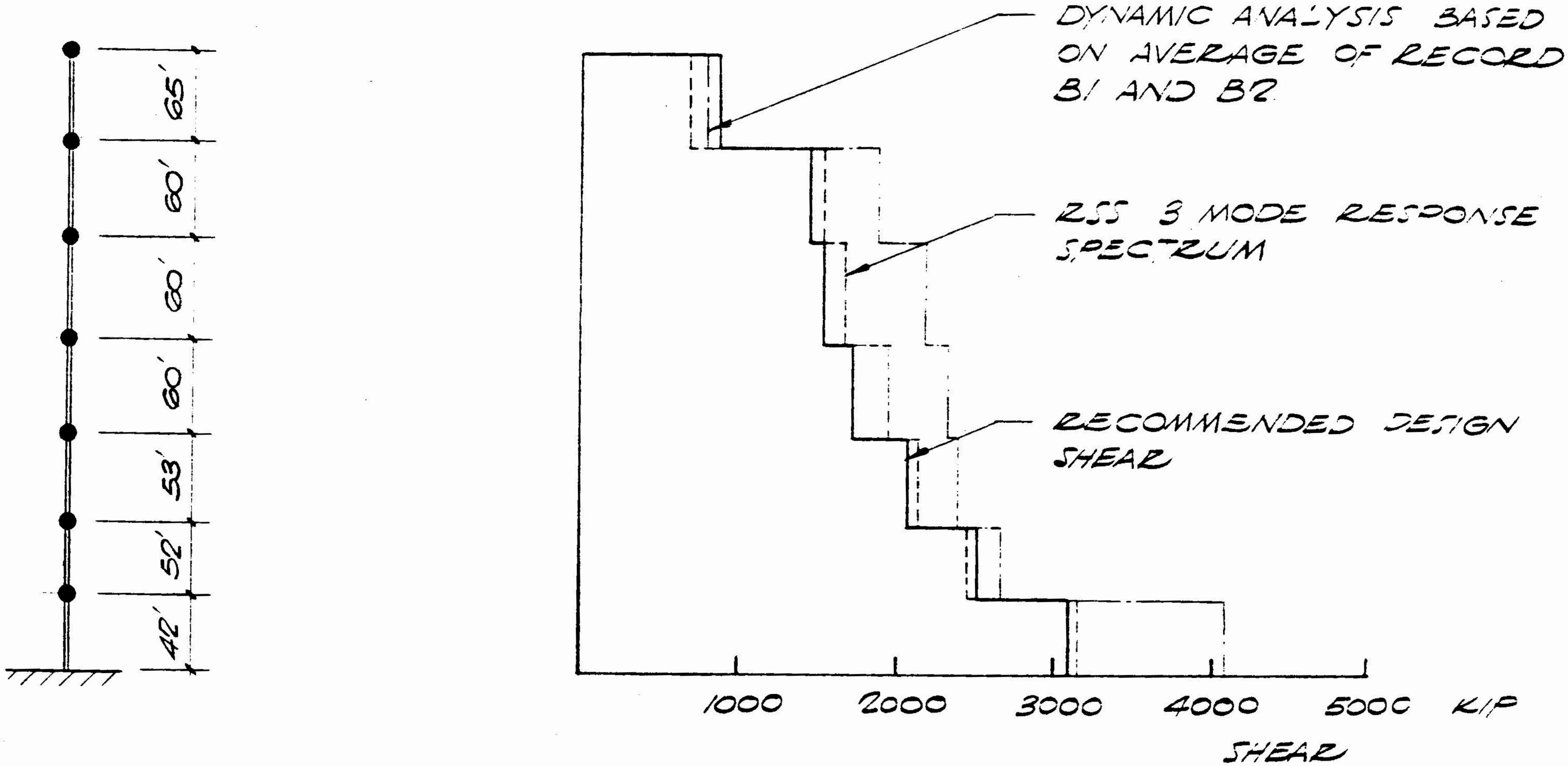
INTERSTOREY SHEAR BASED ON DYNAMIC ANALYSIS

FIG. 13



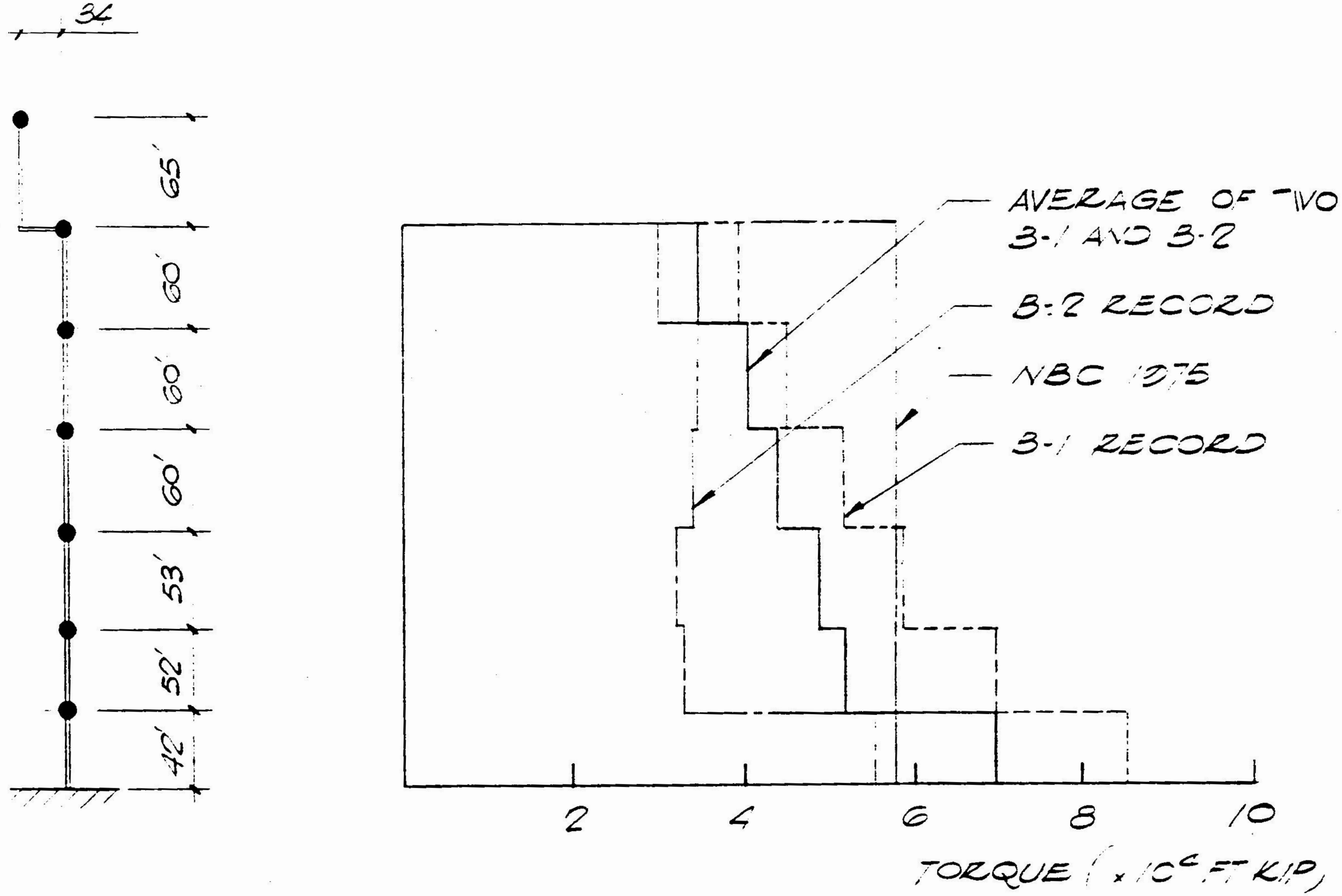
SENSITIVITY OF SHEAR ENVELOPE

FIG 14



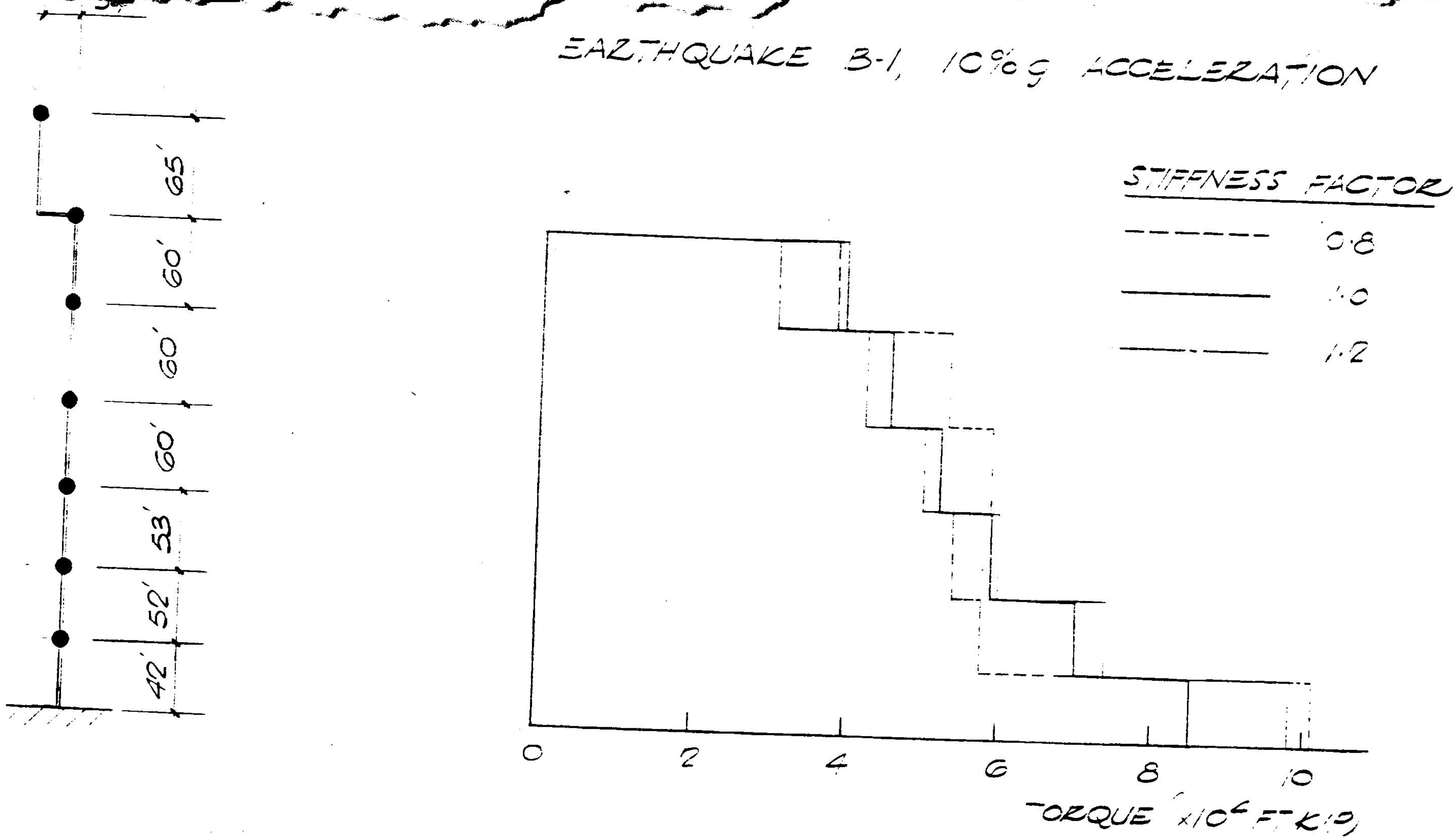
SUMMARY OF INTERSTOREY SHEARS

FIG. 15



TORQUE ENVELOPE FROM DYNAMIC ANALYSIS

FIG. 16



SENSITIVITY OF TORQUE ENVELOPE

FIG. 17

TABLE I: PERIODS OF NATURAL MODES

MODE *	N-S DIRECTION	E-W DIRECTION
1.	3.89 SEC (TORSIONAL)	4.16 SEC (TORSION PREDOMINANT)
2.	3.74 SEC. (FLEXURAL)	3.53 SEC (FLEXURAL PREDOM.
3.	1.34 SEC. (TORSIONAL)	1.39 SEC (TORSION PREDOM.)
4.	1.31 SEC. (FLEXURAL)	1.27 SEC (FLEXURAL PREDOM.)
5.	0.90 SEC. (TORSIONAL)	0.91 SEC (TORSION PREDOM.
6.	0.80 SEC. (FLEXURAL)	0.78 SEC (FLEXURAL PREDOM.)

* MODES ARE NUMBERED ACCORDING TO DECREASING ORDER OF PERIOD.