Seismic analysis and design of the Skytrain cable-stayed bridge

M.S.Khalil & L.H.Bush
Bush Bohlman-Reid Crowther, Vancouver, British Columbia, Canada

ABSTRACT: The Skytrain Fraser River Crossing is a cable-stayed bridge currently under construction near Vancouver, B.C. and is scheduled for completion by late 1988. The bridge is located in a highly seismic area with different soil conditions on the two banks of the river and liquefaction potential in the event of a major earthquake. This paper discusses the analysis and design considerations of the bridge for earthquake effects. Discussed in the paper are site conditions, design criteria, input motion, analysis methods and design parameters.

1 INTRODUCTION

The Skytrain Fraser River Crossing is currently under construction near Vancouver, B.C. and is scheduled for completion by late 1988. The bridge forms a major component of extension Phase II of the automatic light rail transit system which serves the greater Vancouver area. It is intended to carry light rail transit vehicles from New Westminster, where it presently terminates, across the river to the suburb of Surrey.

The bridge is a cable-stayed structure with a centre span of 340 metres flanked by 138 metre spans for a total length of 616 metres between anchor piers. Towers are diamond shaped extending some 120 m above water level. The shafts of the towers are hollow with a wall thickness of 500 mm and are connected at the top by a solid diaphragm to which the cables are anchored. The deck consists of a pre-stressed concrete solid slab spanning between two 1.11 m deep upstand edge beams. The deck has an overall width of 12.56 m and is supported by two planes of cables spaced at 11.0 metres (Figure 1).

The bridge is located in a highly seismic area with a deep seismic source zone within 90 kilometres from the site (Hofman, 1982). Soil conditions are complex and vary substantially from one bank of the river to the other.

The entire site is underlain by bedrock located at a depth of some 30.0 metres on the north side and 80.0 metres on the south side (Figure 1). On the New Westminster side of the river the bedrock is overlain by dense glacial deposits of varying depth while the Surrey side is overlain by a layer of compressible marine silt and dense to loose Fraser river sands.

These subsurface conditions are complex and required careful evaluation of input motion and detailed assessment of foundation behaviour.

The Skytrain Bridge is a high level cable-stayed structure and the manner in which the deck is connected to the towers has a major influence on the behaviour of the bridge. If there is no connection between the deck and towers the seismic inertia forces will be kept to a minimum but the structure may be too flexible under service conditions since all longitudinal loads are transmitted through the cable system to the top of the towers.

The introduction of rubber block bearings between the deck and the towers increases the stiffness of the bridge, thereby reducing service load displacements to acceptable values. Although such a bearing increases the seismic forces from those which occur with no deck to tower connections, they can be kept to reasonably low values by judiciously selecting the stiffness of the bearing.

The deck is supported by two large rubber block bearings at each of the main towers acting in shear under horizontal loads. At the anchor piers the deck is free to slide longitudinally while a shear key on the centreline of the bridge restrains relative movement in the transverse direction.

A detailed study was carried out in the early stages of the design in which various levels of shear stiffness were assigned to the bearings and the behavior of the bridge was evaluated under both service and earthquake loads. On the basis of this study the bearings were dimensioned so that they provide a shear stiffness equal to 10000 KN/m at each tower. This stiffness is high enough to produce acceptable performance under day to day service conditions, yet is soft enough to prevent high seismic inertia forces from being transmitted to the towers from the deck (see bearing detail, Figure 1).

2 DESIGN CRITERIA

As a vital link in the Greater Vancouver transportation network the bridge is designed to survive moderate earthquakes without damage and with almost no interruption in service. On the other hand, economic considerations preclude the same requirements at extreme earthquake levels. Put in more definitive terms, the two design earthquakes are described as follows:

a) Moderate Earthquake: This category covers earthquakes with a return period of up to 100 years, i.e. a probability of exceedance of 1% in any one year. Site seismicity puts the peak ground acceleration for such earthquakes at 12% of gravity. Under this level of seismic event the structure is to remain elastic.

b) Extreme Earthquake: This is an earthquake with a return period of 475 years (i.e. 0.2% probability of exceedance in any one year). Under this level of seismic shock, represented by a peak ground acceleration of 24% of gravity, the structure-foundation system must be capable of enduring the resulting stresses and deformations without the collapse of the bridge or its foundation or any primary structural elements. However, repairable local damage and tolerable inelastic deformations may be sustained.

A specific objective of the designers has been to develop a bridge design in which the seismic requirements do not, in general, dominate the design and therefore the premium for earthquake resistance is kept to a minimum.

3 INPUT MOTION

Ideally a sufficient number of earthquake records should be available which reflect both the tectonic conditions in the area and the effects of the soil formation at the proposed bridge location. For the Skytrain Bridge site, however, earthquake records are lacking. The plan adopted to produce appropriate input motion for the bridge is summarized as follows:

a) On the basis of previous studies of the seismicity of the northwestern regions, and a comparison of earthquakes recorded elsewhere, three base rock earthquake records were selected.

The criteria for the selection of the earthquakes included similarity with the Skytrain bridge site in expected magnitude of earthquake, epicentral distance and soil formation. Three earthquakes were chosen:

- o The San Fernando, California earthquake (CALTECH), 1971
- o The Imperial Valley, California earthquake (CALTECH), 1979
- o The Santiago, Chile earthquake, 1965.

With several earthquakes a wider range of energy content was covered since no single earthquake could be considered representative of the site.

b) In addition to these base rock records, the Olympia, Washington 1949 earthquake surface record was considered to be indicative of surface motion in the northwest region.

c) The earthquake motions propagate through the overburden soil from the underlying rock mainly in the form of shear waves. These motions transfer dynamic forces from the soil to the bridge foundations and thence to the bridge structure itself. In the analysis of the bridge, these motions act in the form of base accelerations applied to the foundations of the bridge. To determine these accelerations from the base rock time histories, one-dimensional shear wave propagation analyses were performed using the computer program (SHAKE). This procedure utilizes the time history at bedrock to produce the free field motion at the ground surface for a given soil profile (Byrne & Atukorala, 1985).

d) The wave propagation analysis was carried out for the soil profiles at both the north and south towers resulting in two sets of surface free field motions. The analysis was based on an extreme earthquake level of 24% of gravity and a moderate level of 12%. A damping value of 5% of critical was used in all analyses.

e) To account for the inherent uncertainty in the determination of soil properties, upper and lower bound values were used to bracket the surface response.

f) Six ground response spectra were produced at each of the two towers for the extreme level earthquake and a similar number for the moderate earthquake.

g) To simplify the design, and accepting the premise that the bridge design would not be controlled by seismic requirements, the designers opted for a single smoothed response spectrum to be used in all seismic analyses. The effects of out of phase motions were evaluated separately. The design spectrum was constructed as an envelope to the response spectra resulting from the wave propagation analyses, particularly in the range of critical periods of vibration (greater than 1.0 second) and disregarding sharp local spikes in the spectra.

Figure 2 shows some of the response spectra produced by the wave propagation analysis as well as the design response spectrum. Also shown is the surface response spectrum of the 1949 Olympia earthquake scaled to 24% of gravity.

4 ANALYSIS OF THE SUPERSTRUCTURE

A three dimensional computer model of the entire bridge was created. The deck was represented by three elastic spines in the longitudinal direction and transverse members at cable connections and supports. Bending and torsional stiffnesses in the longitudinal direction were assigned to the three spines which added up to the properties of the full section. The mass of the deck slab was assigned to the transverse members which, together with the mass of the edge beams, gave an accurate distribution of the deck inertia. Superimposed loads were represented by additional masses.

The cables were represented by straight members in this analysis. Their stiffnesses were calculated on the basis of the forces under full dead loads using a modified Ernst's equation for a parabolic cable.

Linear elastic response spectra analyses were carried out in both the longitudinal and the transverse direction using the design response spectrum corresponding to the extreme earthquake with a ground acceleration of 24% of gravity. Response values due to a moderate earthquake with an acceleration of 12% of gravity are 50% of the extreme values. The three dimensional model proved to be particularly useful in the transverse direction where the forces in all support structures were obtained in a single run. On the basis of preliminary analyses, the lowest fifty modes in the transverse direction and the lowest thirty modes in the longitudinal direction were retained for response calculation. Table 1 gives the periods and type of vibration for the first twelve modes. The flexibility of the bridge is apparent from the fact that the predominant modes have periods in the range of 3 to 7 seconds. Since most of the energy input of the earthquake lies in the range of period of 0.5 to 1 seconds, it was reasonable to expect low inertia forces due to seismic loads.

The square root of the sum of the squares (RMS) method was used to calculate the response values, provided the modes were separated sufficiently to preclude the possibility of their simultaneous occurrence. This separation was considered acceptable if the periods of the relevant modes were different by more than ten percent, otherwise a method using the peak plus the square root of the sum of the squares of the remaining modes was used.

Figure 3 shows a graphical representation of some response values. The vertical axis represents the value of response as a percent of the maximum values obtained in the analysis. The horizontal axis shows the number of modes used in the RMS calculation. The figures illustrate that while some maximum values of response are obtained by combining the first few modes, some other values require a large number of modes to calculate their maxima.

Some of the results of the analysis are summarized in Table 2 corresponding to the extreme earthquake with a ground acceleration equal to 24% of gravity. The total base moments are the overturning moments at the top of the foundation piers. The base shear values in terms of dead load reaction were determined as 0.047 W and 0.056 W in the longitudinal and transverse directions respectively. These values are fairly low, reflecting the flexibility of the bridge.

Moments along the tower leg are given as absolute values and in terms of the ultimate moment capacity of the particular section at the same axial force. A resistance factor of 0.5 was included in the ultimate moment values in accordance with ATC guidelines (ATC, 1981). Moments due to the moderate design earthquake with a return period of 100 years are 50% of these values. These results and their implications on the design of the towers are discussed further in the section on design and detailing.

5 ASSESSMENT OF NONLINEAR EFFECTS

As previously stated, linear elastic methods were used in the seismic analysis. The second order effects were evaluated separately as follows.

Two single mode (static) analyses were carried out using the computer program COBRA (Khalil, 1985). One analysis was linear while the second included the effects of cable change of geometry (sag effect) and the second order effects due to combined axial load and bending moments in the tower and deck elements. Cracked section stiffness equal to 40% of gross was used for all concrete members. Longitudinal loads equal to 4.7% of the weight of the elements were applied at member end joints. This magnitude was selected to give the same base shear obtained from the response spectra analysis (see Table 2).

Ratios of nonlinear to linear results for some response values were as follows:

Response	Magnification factor
Tower base moment	1.26
Deck horizontal deflection	1.39
Tower top deflection	1.40

The analysis indicated that the lateral loads representing the seismic inertia forces introduce very low axial forces in the deck and tower elements compared to the dead load forces. This implied that nearly all the second order effects were due to the presence of axial loads from dead load forces and the same magnification due to second order effects is therefore valid for any lower earthquake loading. The nonlinear forces due to the moderate earthquake were consequently 50% of the extreme values.

6 EFFECT OF LONG SPAN LENGTH ON INPUT MOTION

In the usual treatment of earthquake excitations an assumption is implicitly made that the same motion acts at all points of the structure's foundation. For translational type of motion the bedrock is assumed to be infinitely rigid, an assumption which does not comply with the concept of seismic wave propagation through the earth's crust from the point of fault rupture. However, in most practical applications the base length of the structure is small relative to the vibration wave lengths of the bedrock and this assumption is therefore acceptable (Clough & Penzien, 1975).

The main span of the Skytrain Bridge is 340 m and the span between end piers is 616 m. A wave propagating at a velocity of 2000 m/sec. with a frequency of 3 Hz (a period of 1/3 sec.) will have a wave length of 670 m. This wave will obviously result in significantly differing motions along the length of the bridge.

There have been no measurements made during an earthquake of such relative movements at different points on the earth's surface, however it is clear that they must take place and it is important that future methods of analysis be developed to allow different earthquake motions to be applied at different supports. Available methods of seismic analysis have not yet progressed to the stage of providing a direct solution to this problem and approximate approaches must be used.

The approach used herein utilizes a static analysis of the bridge in the longitudinal direction. The maximum free field motion that may occur in the ground during an earthquake with a return period of 475 years was conservatively estimated as 300 mm at the north tower. Since the structure is so flexible, it is reasonable to assume that it can transmit little energy into the soil. The free field motion will not therefore be altered markedly by the structure and the 300 mm therefore provides an adequate measure of the foundation displacement.

The surface response spectrum provides another source of foundation displacement. The maximum relative displacement at the period of the first longitudinal mode of vibration of the structure was found to be 375 mm. It was considered that a relative displacement between the main towers equal to twice this movement constituted a conservative estimate of the possible out of phase motion.

The moment at the base of tower N1 due to a horizontal foundation displacement of 750 mm was calculated as 111,000 KN.m. This moment is well below the moment obtained from the response spectra analysis and therefore did not present a critical design condition.

7 DESIGN AND DETAILING

The main towers at N1 and S1 are the major elements to resist longitudinal and transverse loads from wind and seismic effects with the anchor piers N2 and S2 attracting a minor share of the total lateral loads. The design criteria specified that the bridge was to remain elastic under the moderate earthquake while some inelastic deformations could be tolerated under extreme earthquake conditions. As previously mentioned, the forces due to the moderate earthquake are 50% of those due to the extreme earthquake. A ductile design under the extreme event in accordance with the ATC provisions would be based on reduced forces obtained by dividing the elastic forces resulting from the analysis (Table 2) by a reduction factor accounting for ductility. In a frame type structure, such as the tower, the reduction factor for moments may be as high as five. But any factor greater than two would result in forces lower than the forces due to the elastic (moderate) earthquake. It is therefore evident that if the towers are designed as ductile frames the requirements of the elastic earthquake are more stringent than the extreme forces for any ductility factor greater than two.

The ATC provisions recommend that seismic loads be combined with other appropriate loads in an ultimate load condition using a load factor of 1.0. On the other hand wind loads are factored by 1.3 for ultimate strength design. Table 3 gives the tower base moments due to the elastic (moderate) earthquake and factored wind loads. Comparing the two sets of values, it is apparent that wind forces generally control the design. However, detailing of reinforcement, particularly with respect to confinement of reinforcing was generally governed by seismic design requirements.

With reference to Table 2, the moments at all critical sections along the tower leg due to the elastic earthquake (50% of the values shown) are lower than the ultimate moment capacities at the particular sections. This implies no ductility demands on any of the tower sections in order to resist the moderate earthquake. Even under the extreme earthquake forces the capacity of the tower shaft sections above the deck was not exceeded. Plastic hinging of these sections is therefore not expected and confinement reinforcement was not required. These sections were reinforced for shear and as such have sufficient steel that, even in the unlikely event of plastic hinging, there would be moderate confining of the section. Some sections below the deck would be subject to yielding in the extreme earthquake (Table 2), but the ductility demands are very low.

The low ductility demands permitted some liberty in detailing the tower sections. Only corner elements of the tower shaft cross sections were confined. The sizes of these elements were selected such that their confined cores provide adequate support for the dead weight of the structure after the remainder of the section has spalled. Confined regions are shown on the tower elevation in Figure 1. The maximum spacing for confinement steel was set at 150 mm in lieu of the 100 mm limit recommended by the ATC guidelines. This selection is supported by recent research conducted at the Portland Cement Association (Oesterle et al 1981) and is more stringent than the 200 mm permitted by the New Zealand Code (NZS, 1982).

To account for the possibility of simultaneous occurrence of earthquake shocks in more than one direction, seismic forces in the longitudinal and transverse direction were combined such that 100% of the earthquake in one direction was considered to act simultaneously with 30% of the earthquake in the other direction.

8 FOUNDATIONS

The two main towers and the anchor piers are all supported by pile foundations. On the south side of the river at both S1 and S2 steel friction pipe piles 610 mm in diameter are driven closed end and filled with concrete. The piles are located in the river sand where liquefaction of the upper layers was predicted in the event of a major earthquake. A network of timber compaction piles surround the foundation at each of the two locations to prevent liquefaction of the upper sands.

Pile design at S2 is controlled by earthquake loads. The piles are conservatively designed such that their useable ultimate load, defined as the product of the resistance factor and the ideal strength, is not exceeded when subjected to forces from the extreme earthquake. Piles are anchored to the foundation by reinforcing steel with sufficient length inside the pile to ensure tensile load transfer to the concrete and then by bond to the steel wall of the pile.

Pier S1 is founded in the river and pile design is controlled by forces from ship collision which are much higher than seismic forces.

On the north side of the river both piers are founded in relatively deep water. The foundation at N2 utilizes steel H piles founded in dense glacial material to provide additional protection against scour and to aid in resisting overturning moments by developing tensile forces in the piles. Here too, the ship collision forces govern the design of the piles.

Of the three river piers, N1 is the most vulnerable. It is founded in deep water in medium dense sand. The pier is supported by 915 mm diameter steel pipe piles driven open end into rock, some 28 metres below the base of the foundation, and filled with concrete. The piles are extended through a 10 metre tremie concrete base and terminated into a structural concrete footing. They are assumed to be fixed at the underside of the tremie concrete and adequate reinforcement is provided in the tremie around the piles.

The foundation was designed for ship collision forces and then checked for seismic loads. The ship collision design required a massive concrete pier at the waterline to withstand the impact from a colliding ship. The inertia forces associated with this pier in the event of an earthquake were a source of concern. The upper sands are subject to liquefaction in the event of an earthquake which could result in loss of lateral support of the vertical piles. Geotechnical investigations resulted in an estimated depth of liquefaction to 10 metres below the base of the pier.

Computer analyses were carried out on a plane frame model of the pier and supporting piles. In the model, shown in Figure 4, the bending stiffness of the piles were lumped in a single line of elements fixed to the base of concrete and extending down to rock level. Horizontal linear springs were connected to the bending elements to represent the lateral soil support. The stiffness of these springs was determined by a geotechnical investigation which accounted for the properties of the soil layers at N1 and the cyclic nature of the seismic motion. Vertical axial load carrying members were used at pile locations to represent the axial stiffness of the piles. The superstructure was represented by a spring-mass unit attached to the top of the pier. The mass and spring stiffness were determined from the dead load supported by the foundation and the characteristics of the first mode of vibration of the bridge in the direction under consideration.

A depth of liquefaction of 10 metres was assumed and no lateral soil support was provided to the piles along that depth. Lumped masses were used to represent the added inertia of the surrounding water.

Three separate analyses were performed on the foundation model:

a) A response spectra analysis using the design response spectrum corresponding to the extreme earthquake (24% of G).

b) A response spectra analysis using the N-S component of the 1940 El Centro earthquake scaled to 0.24 G.

c) A time history analysis using the El Centro earthquake scaled to 0.24 G.

Some of the results of the analysis are shown in Figure 4. Inspection of these and other results, and comparison with loads from ship collision analyses, led to the following conclusions.

a) The analyses were carried out for the extreme earthquake, with a peak ground acceleration of 24% of gravity. At this level of earthquake the forces are well below the ultimate capacity of the pile section, although some yielding may occur. Under a moderate earthquake (12% of gravity) the forces were below the first yield capacity of the pile.

b) The seismic acceleration used in the analysis, 0.24 G, is the acceleration at ground surface. A more appropriate value for the response spectra may have been the acceleration at the base of the concrete pier, estimated by the geotechnical investigators to be approximately 70% of the ground acceleration. This adds a measure of conservatism to the results of the response spectra analyses described above.

9 CONCLUSIONS

The detailed seismic analysis and design of the Skytrain cable-stayed bridge has been presented. The determination of input motion, the treatment of second order effects and detailing for ductility have been described. It has been shown that by properly selecting the bearing conditions a degree of isolation was achieved for the inertia of the deck and the seismic forces are so controlled that they no longer govern the design. By detailing for ductility the bridge was made capable of withstanding the forces from more severe seismic events than the design earthquakes.

ACKNOWLEDGEMENT

This paper is published by permission of the British Columbia Transit Corporation. The authors express their appreciation to Mr. H. Zimmerling, Chief Structural Engineer for BCT during the design phase of the bridge.

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Table 1 - Modes of vibration

Mode Number	Period (sec)	Type of Vibration
1	6.55	TD
2	6.07	LO
3	3.71	BS
4	3.11	BA
5	3.01	BA
6	2.28	TT
7	2.21	TT
8	1.92	TD
9	1.89	BS
10	1.65	BA
11	1.54	BA
12	1.49	TO

TD = Transverse, Deck
LO = Longitudinal
BS = Bending, symm.
BA = Bending, antisym
TT = Transverse, towers
TO = Torsional

Table 2 - Maximum values of response
[Extreme earthquake, 0.24 G)

RESPONSE	Direction of Earthquake Longitudinal	Transverse
Total tower base shear, KN	5640 (.047 W)	5760 (.056 W)
Total tower base moment, KN.m	264000	538000
Tower leg moments:		
At base	132000 ($.825\ M_u$)	169000 ($1.35 M_u$)
Below deck cross beam	37000 ($0.35\ M_u$)	56300 ($1.17\ M_u$)
Above deck cross beam	37000 ($.36\ M_u$)	36900 ($.72\ M_u$)
Below top diaphragm	18000 ($0.18\ M_u$)	38000 ($.86\ M_u$)
Deck displacement, mm		
Horizontal	400	545
Vertical	108	-
Tower top displacement, mm	428	280

Table 3 - Ultimate tower base moments

Loading	Longitudinal Moments (KN.m)	Transverse Moments (KN.m)
Wind loads	111700	106000
Seismic, 100 yr.	66000	84500

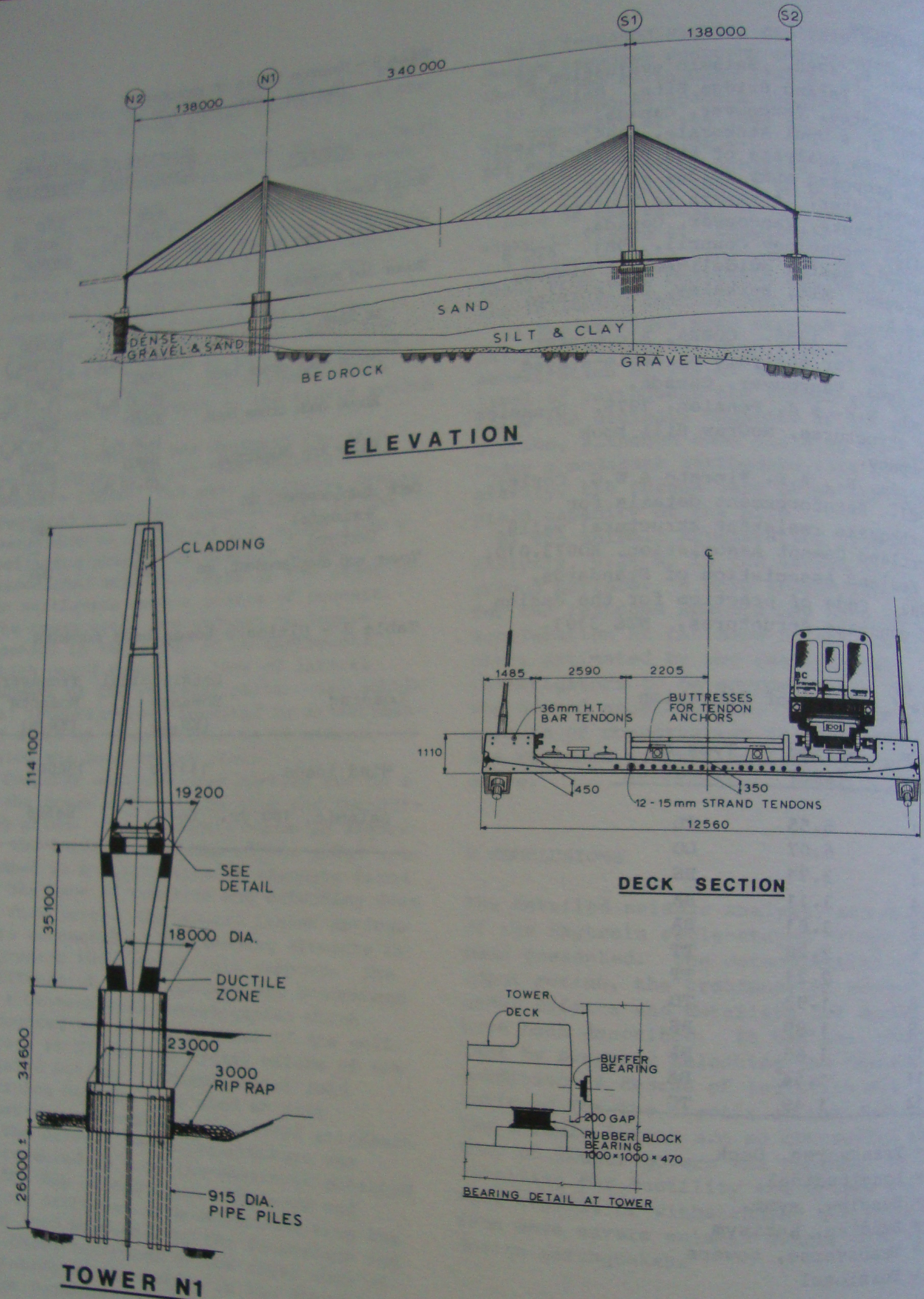
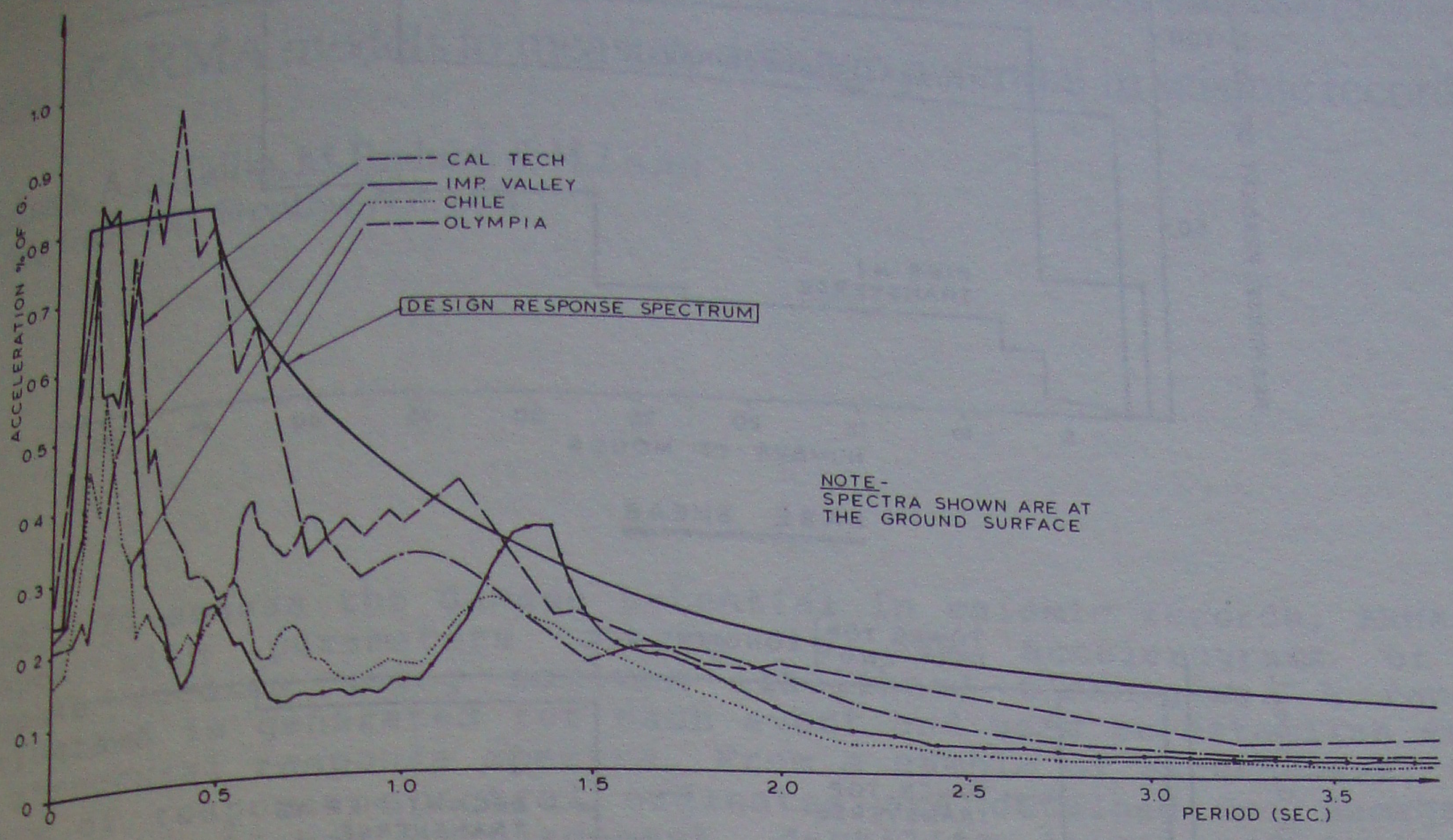
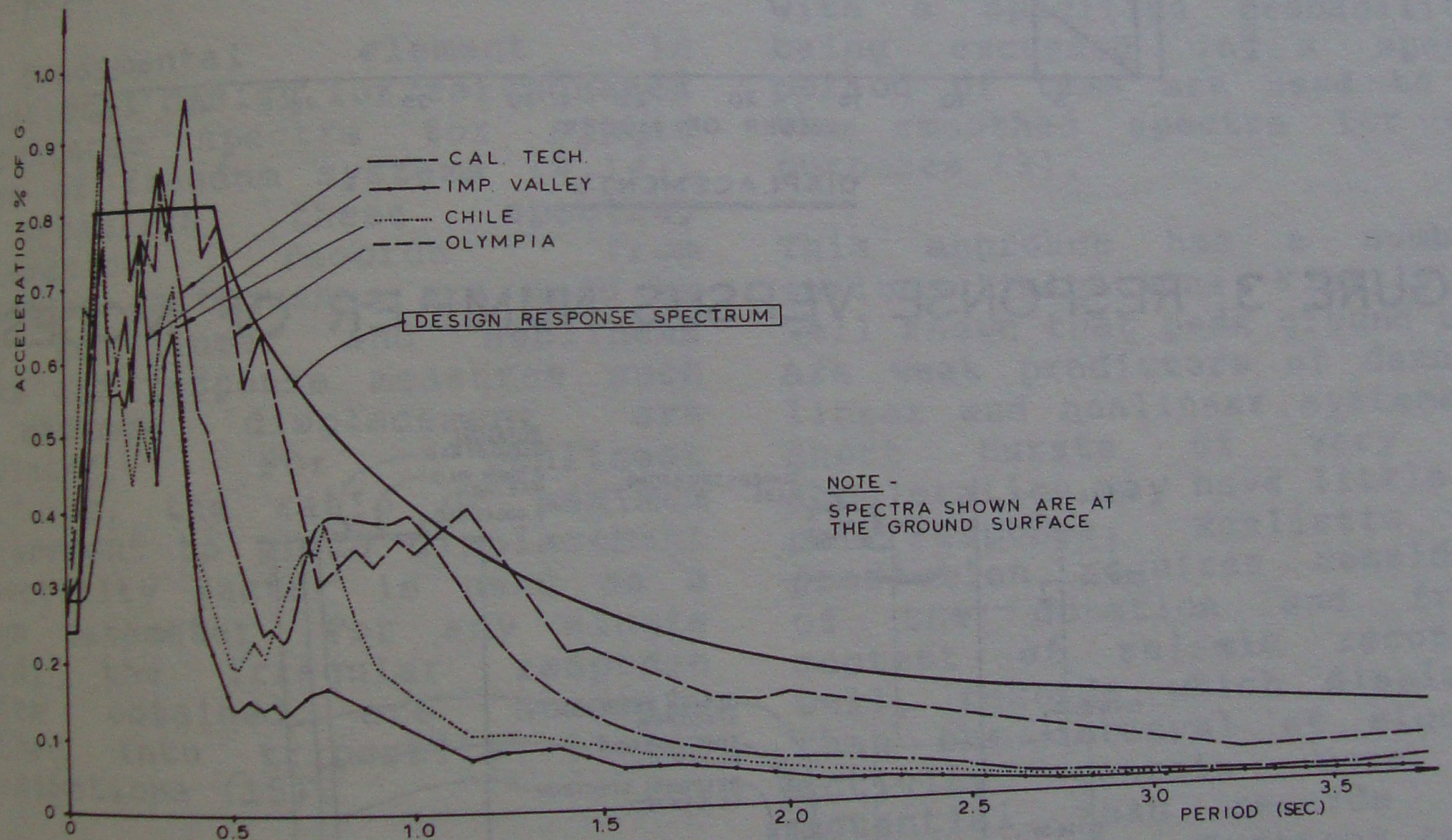


FIGURE 1. GENERAL ARRANGEMENT



AT S1



AT N1

FIGURE 2. ACCELERATION RESPONSE SPECTRA

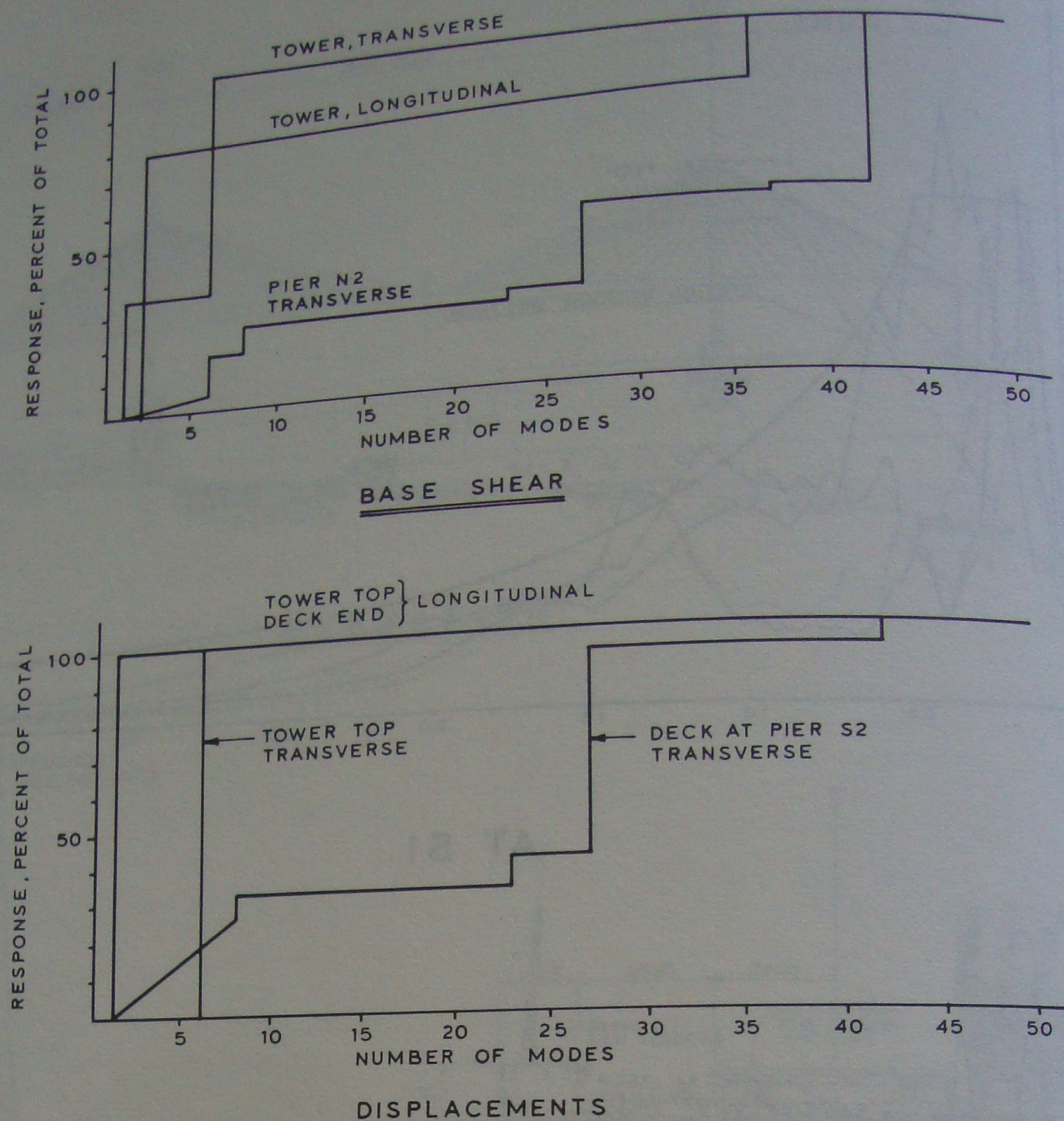


FIGURE 3. RESPONSE VERSUS NUMBER OF MODES

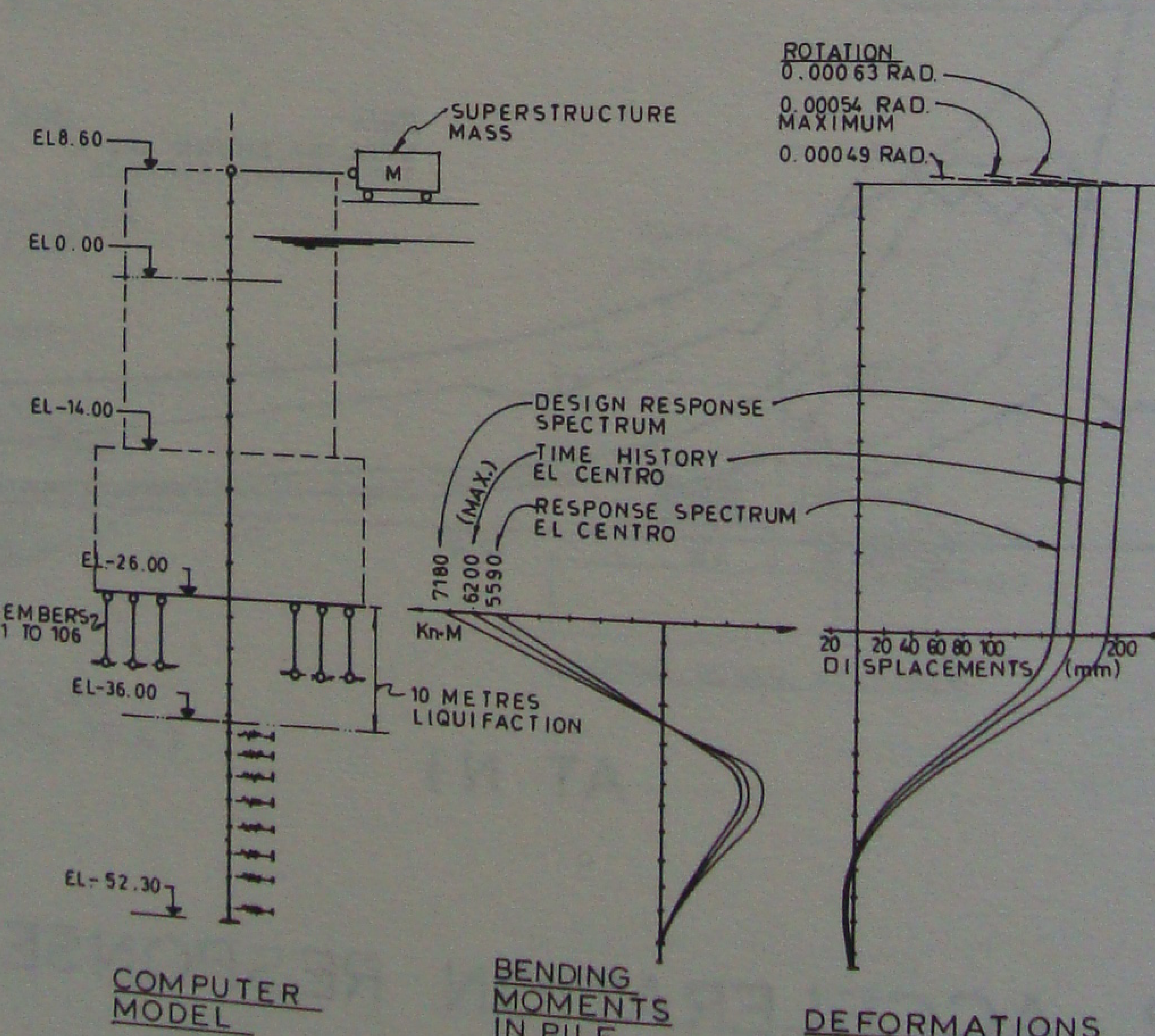


FIGURE 4. N1 PILED FOUNDATION ANALYSIS