EARTHQUAKE RESPONSE AND EARTHQUAKE RESISTANT DESIGN OF LONG SPAN SUSPENSION BRIDGES

Ichiro Konishi* and Yoshikazu Yamada**

ABSTRACT

Natural periods of suspension bridges, earthquake response of suspension bridges due to actual earthquake, and a methods of analysis of suspension bridge towers are given in this paper continue to previous report. Related with these points of view methods of earthquake resistant design of long span suspension bridges are discussed and summarized at the conclusion of this paper. The methods of analysis in this paper are similar to the analysis of the previous paper, and some improvements are introduced.

INTRODUCTION

A method of analysis of earthquake response of long span suspension bridges was developed and reported at the II WCEE.(1) The fundamental dynamic characteristics of the suspension bridge to earthquake were investigated in the paper. The suspension bridge was simplified into a physically analogous system, and natural frequancies and responses to a simple ground motion were obtained.

Earthquake ground motion is applied to the suspension bridge through two tower bases and two anchorages. These points are subjected to quite complicated dynamic effects of earthquakes. In long span suspension bridges, moreover, the combination of deep piers, large anchorages, tall towers, long cables, and suspended structures, of various rigidities, results in unusual problems in the calculation of earthquake response.

Since the towers and the anchorages are connected by cables, ground acceleration is not enough for the computation of earthquake response, and information on ground displacement is required. The strong motion recording programs are mainly limited to record the acceleration, and there arises another difficulty in the analysis of suspension bridges.

* Professor of Kyoto University, Department of Civil Engineering
** Assistant Professor of Kyoto University, Department of Civil Engineering

In this paper, natural frequencies of the suspension bridge are obtained using the methods of analysis given in previous paper, and, then, the earthquake response due to an actual earthquake will be computed by using a high speed computer. Methods of analysis of suspension bridge towers will, then, be discussed as it is the most important for the earthquake resistant design. In conclusion of this paper outlines of the methods of earthquake resistant design will be disscussed using the results obtained in this paper.

Numerical computations in this paper will be done using a long span suspension bridge which is now being planned at Seto Naikai inland sea connecting Honshu and Shikoku in Japan. The methods and the results of the analysis given in this paper are also applicable to other long span suspension bridges.

NATURAL PERIODS OF THE SUSPENSION BRIDGE

Long span suspension bridge are consist of the structural part with various rigidities, and the natural periods of the bridge, therefore, are quite important to obtain the rational methods of earthquake resistant design of the bridges.

As the long span suspension bridges are composed of structural parts with various rigidites the separation of the system into some idealized sub-systems is required to obtain the agreeable numerical results because of the limited capacity of the computer and of the stability of the computation. In this paper, the natural periods of the suspension bridge of the various systems are obtained assuming proper boundary conditions.

The numerical computation of the analysis will be done on the proposed Akashi Straits Bridge at Seto Naikai in Japan. It has the following dimensions.

Center span length	1,300 m
Side span length	650 m
Center to center distance of the trusses of suspended structure	24 m
Height of stiffening truss	14 m
Moment of inertia of the trusses	2 X 4 m^4
Maximum cable sag	108 m
Sag ratio	1/12
Heigth of towers	200 m

Deal loads	20 ton /m
Weight of both towers	30,000 ton
Horizontal component of cable tension due to deal load	19,560 ton / cable
Diameter of pier	50m
Depth of pier	50m

(a) Vibrations of Superstructures in Longitudinal Direction

The first system is that shown in Fig. 1. It is assumed that the suspended structures vibrate in the vertical direction, the cable vibrate in the vertical and horizontal directions, and the tower in the horizontal direction. This motion may occure when an earthquake acts at longitudinal direction to the bridge. In this case, the suspension bridge is idealized as a finite degrees of freedom system as shown in Fig. 1. The method of analysis and some numerical results for the system were given in the previous paper.(1) The systeme of Fig. 1 is 29 degrees of freedom and has more accurate analogy than the system of previous paper.

The natural periods and their modes obtained are shown in Table 1 and Fig. 2. The higher mode vibrations are fairly distorted due to idearization of the system and omitted in the Fig.2. Remarkable coupling motion of suspended structures and the towers occurs only at the first symmetric mode as shown in Fig. 2.

(b) Vibration of the Towers and Piers.

According to the preceding analysis, coupling motion of the towers and the suspended structures is not important for higher mode vibrations where the deflections of the towers are predominant. The analysis considering the tower and the proper spring constant at the top of the tower are assumed is, therefore, approximately possible. Using the method of analysis of towers which will be explained lately the natural periods of the tower are obtained as shown in Table 2, Column (1).

Stiffness of the piers is quite large compared to other structural parts of suspension bridges and considered to be rigid in preceding investigations. Table 2, Column (2) shows the natural periods of the pier having shearing stiffness. No superstructures are assumed in this case.

In Column (3), the natural periods of the system including the pier, the tower, and the proper spring constant at the top of the tower are shown and compared with Column (1) and (2).

No effects of surrounding water around the pier are considered in these analysis.

(c) Lateral Vibration of Cables and Suspended Structures

Lateral vibration of the Cable and suspended structure system are investigated by the same kind of structural aystem of Fig. 1. except the followings.

(1) Only the center span structure is considered assuming the fixed supports at the tops of the towers.
(2) Masses of the cables and suspended structures are considered to be separated to each of them and pendulum action of the cable and suspended structure system is considered.
(3) The center span is divided in 16 segments in stead of 8 segments. The syetem such considered has 32 degrees of freedom.

The first few natural periods and their modes are shown in Table 3 and Fig. 3. As shown in this figure, in lower modes, the system vibrates like a double pendulum, in higher modes, however, the cable and the suspended structure vibrate rather independently, and there is no coincidence of nodal points of the cable and the suspended structure.

EARTHQUAKE RESPONSE

Earthquake response of the suspension bridge due to the ground motion in the longitudinal direction were computed. The earthquake used is the displacement record of the 1957 So. California Earthquake shown in Fig. 4. It is a tipical earthquake having one large shock wave at the biginning of the quake, and its characteristics were discussed by Prof. Housner and Prof. Hudson.[2]

Modal analysis given in previous paper[1] was used in the computation, and Newmark's β method was applied to step-by-step integration. A time interval in the numerical integration is selected as $h = 0.018939$ sec and the dynamic response was printed out at every three time intervals.

Time response curves are shown in Figs. 5 and 6. Figs. 5(a) and (b) are the responses for the ground motion applied to the left side anchorage, and Fig. 6 (a) and (b) are for the ground motion applied to the base of the left side pier.

The incremental stresses due to the earthquake are computed as shown in Table 4. It is clear from Table 5 that the stresses in the suspended structures are not important. The stress in the cable is not small compared to the stresses in the suspended structures, but the allowable stress of the cable is fairly large compared to that of ordinary steel. The stresses in the tower are comparatively large.

ANALYSIS OF TOWERS

From the results of previous paper(1) and the earthquake responses given above, it is concluded that more careful considerations to the earthquake resistant design of towers than that of suspended structures and of cables must be given. The failure of the towers leads to the collapse of the suspension bridge, while the partical damage of the suspended structures does not impair the stability of suspension bridges.

According to previous investigations, the tower could be apploximately analysed independently assuming proper boundary conditions.

Fig. 7 shows schematically the tower to be analysed. Same assumptions used in previous analysis are used in this case and the tower top is restrained elastically be the main cables. It is also assumed that the top of another tower and the anchorage accelerate equally as the base of the tower in order to use acceleration records for the analysis of earthquake response.

With these assumptions the equations of motion for the lateral vibration of the tower is

$$m_i \ddot{z}_i - \frac{1}{b}(M_{i-1} - 2M_i + M_{i+1}) + \frac{P}{b}(z_{i-1} - 2z_i + z_{i+1}) = 0 \qquad (1)$$

$$(i = 1, 2, .., 7)$$

where, z_i = absolute deflection of ith point, M_i = bending moment of the ith point, P = axial force due to cable reaction, b = length of each segment of the tower. For the top of the tower, one must take into account the influence of the restraint of tower by main cables, then the following equation will be obtained:

$$m_8 \ddot{z}_8 - (M_7/b) + \frac{P}{b}(z_7 - z_8) + Ky_8 = 0 \qquad (2)$$

where K may be approximately be given by

$$K = (E_c A_c / L_E) + (E_c A_c / L_{E1}) \qquad (3)$$

In Eq. (3) A_c and E_c are cross section area and Young's modulus of the main cable respectively, and L_E and L_{E1} are given as

$$L_E = \int_0^{l} \frac{dx}{\cos^3\phi} \quad , \qquad L_{E1} = \int_0^{l_1} \frac{dx}{\cos^3\phi_1} \tag{4}$$

where ϕ and ϕ_1 are inclinations of the main cables of the center and side spans. Eq. (3) was obtained under the asumption that the vibration amplitudes of the suspended structures are assumed to stand still like a mass of a displacement vibrograph.

Bending moment M_i in Eqs. (1) and (2) is

$$\begin{aligned} M_i &= -\frac{B_i}{b}(z_{i-1} - 2z_i + z_{i+1}), \quad (i = 1,2,..,7) \\ M_o &= -\frac{2B_o}{b}(z_1 - z_o) \end{aligned} \tag{5}$$

where B_i is elastic constant, selected so as to satisfy the physical conditions.

Substituting Eq. (5) into Eqs. (1) and (2), and taking into account the relation

$$y_i = z_i - z_o \qquad (i = 1,2,...,8) \tag{6}$$

one will obtain a system of differential equations of 2nd order in the form of

$$[A]\ (\ddot{y}_i) + [B]\ (y_i) = (Z) \tag{7}$$

In Eq. (7), square and round brackets, [ ] and ( ) show a square symmetric and a vector matrix, respectively. y_i is the displacement of the ith mass relative to the base of the tower.

For the analysis of Eq. (6) conventional analysis for earthquake resistant design, such as the method of average velocity spectrum [3] could be applied as the external force is the acceleration (Z) instead of displacement in the previous analysis.

The natural periods of the system were already given in this paper, and the first few modes of vibrations are shown in Fig. 7. There are fairly good agreement between these and the results of the system where the cables and the suspended structures are considered.

Using the velocity spectrum of the 1957 So. California Earthquake, the maximum responses were approximately computed as

$$(R) = \sum \left| \text{max. modal response} \right| \tag{8}$$

or

$$(\bar{R}) = \sqrt{\sum \text{(max. modal response)}^2} \qquad (9)$$

The maximum response for bending moment computed by Eqs. (8) and (9) are obtained as shown in Fig. 8. In Fig. 8 the maximum bending moments obtained from response curves of Figs. 5 (a) and 6 (a) are also shown. There are fairly good coincidence between these and $(\bar{M})$'s.

EARTHQUAKE RESISTANT DESIGN OF SUSPENSION BRIDGES

In the followings, methods of the earthquake resistant design will be discussed refering the results obtained and summarized.

(1) For the design of the pier and the anchor blocks, methods of seismic coefficient may be approximately applicable. But in these cases the forces from the towers, the cables, and the suspended structures must be properly estimated since they are rather flexible. Results of dynamic considerations may be useful in this case. The effects of surround water and silts around the piers and the anchor blocks must be properly estimated.[4] For deep piers with less stiffness dynamic consideration must be done.

(2) Dynamic considerations are necessary for the design of the towers. In these cases, the bottom of the tower is possibly considered as a rigid support. For the design of the towers, methods of velocity spectrum are applicable for convenience sake.

(3) Earthquake ground motion with fairly large periods must be auccessively applied to yield large stresses in the cables and the suspended suctuctures. Possibilities of such resonant conditions might be very small.

(4) Seismic coefficient methods may not be applied tothe lateral direction of the cables and the suspended structures as they are very flexible. This relieve the difficulties of the design of lateral strength of the suspended structures and towers. For the analysis of higher mode lateral vibrations of the cables and the suspended structures, they are apploximately treated as indivisual structures.

(5) Directions of the motion of the structures and of the earthquake were restricted to specific directions in this paper, but the earthquake will attack the structures in any direction. Since more precise analysis are required for estimation of these effects, they are now being investigated.

REFERENCES

(1) Ichiro Konishi and Yoshikazu Yamada,"Earthquake Responses of a Long Span Suspension Bridge" Proc. of II WCEE, Vol. 2, pp. 863-878, 1960

(2) G. W. Housner and D. E. Hudson," The Port Hueneme Earthquake of March 18, 1957" Bulletin of the Seismological Society of America, Vol. 48, pp. 163-168, 1958

(3) "The Agadir, Morocco EARTHQUAKE" Committee of Structural Steel Producers of Americal Iron and Steel Institute, Chapter 4, Earthquake Engineering pp. 53-80, 1962

(4) Hisao Goto and Kenzo Toki "Vibrational Characteristics and Aseismic Design of Sub-merged Bridge Piers" Paper presented this Congress (III WCEE)

Table 1 Natural Periods (1) (sec)

modes	symmetric modes	antisymmetric modes
1	11.610	9.207
2	6.219	8.879
3	4.451	4.530
4	4.397	4.397
5	3.659	3.200
6	3.052	3.066
7	2.948	1.299
8	1.267	0.486
9	0.463	0.275
10	0.252	0.190
11	0.170	0.144
12	0.134	0.115
13	0.114	0.093
14	0.093	0.074
15	0.074	

Table 4 Maximum Response Stresses (kg/cm^2)

sections	due to 1957 earthquake applied to	
	anchorage A	tower base B
stiffening truss		
2	3.6	0.32
15	2.2	0.36
tower		
6	51.2	193.7
8	41.1	128.7
10	23.4	132.5
B	45.3	164.9
cable	less than 90	

Table 2 Natural Periods (2) (sec)

modes	(1) tower	(2) pier	(3) tower+pier
1	1.287	0.098	1.287
2	0.471	0.037	0.471
3	0.260	0.024	0.261
4	0.178	0.021	0.178
5	0.140		0.140
6	0.114		0.114
7	0.093		0.099
8	0.074		0.093
9			0.074
10			0.037
11			0.024
12			0.021

Table 3 Natural Periods Lateral Vibration (sec)

modes	symmetric	antisymmetric
1	23.006	13.416
2	8.522	5.474
3	4.765	2.755
4	3.756	2.549
5	2.148	

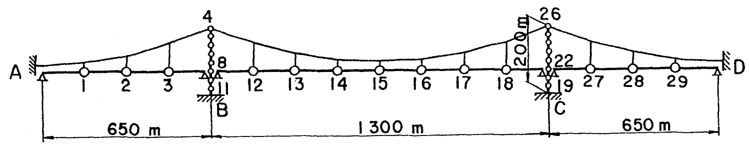


Fig. 1

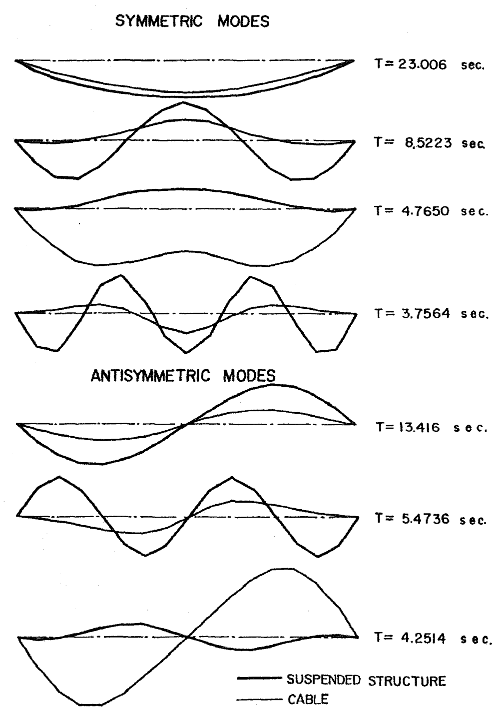


Fig. 3

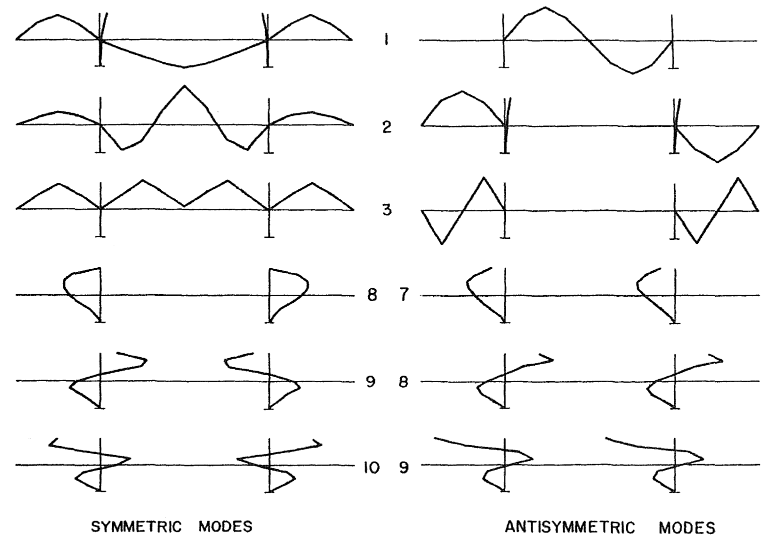


Fig. 2

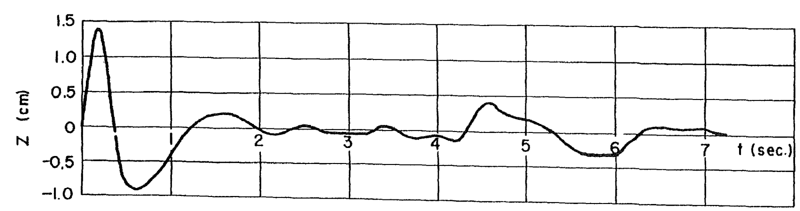
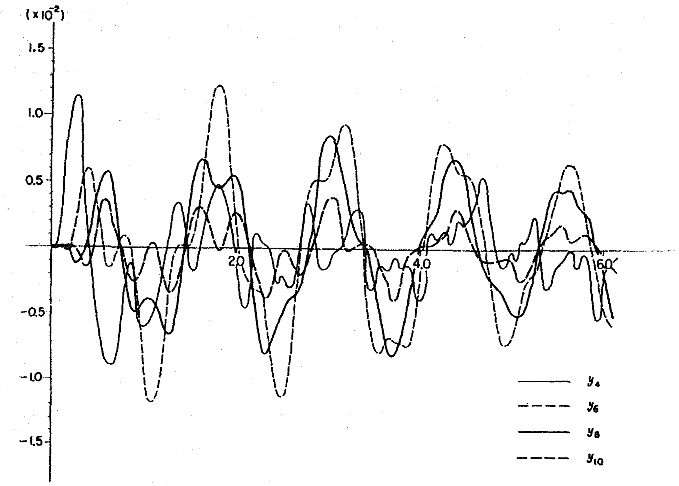
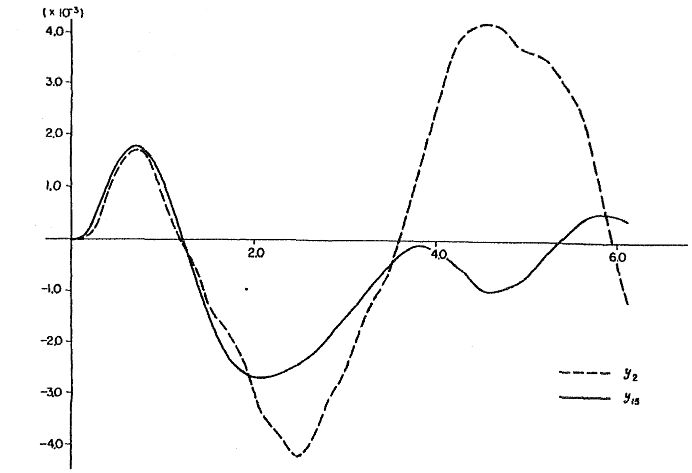


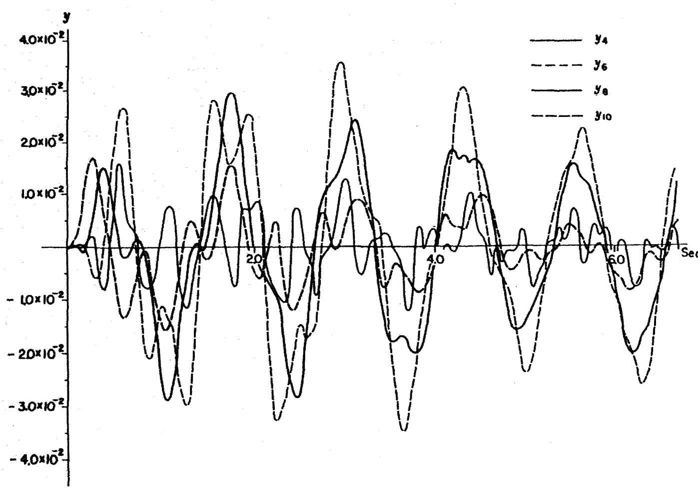
Fig. 4 1957 SOUTH CALIFORNIA EARTHQUAKE
RECORDED AT PORT HUENEME, CALIFORNIA
NAVY RESEARCH AND EVALUTION LABORATORY
MARCH 18, 1957



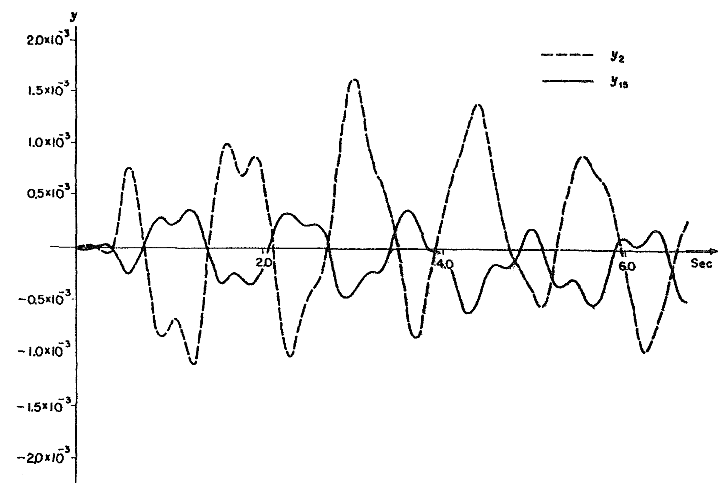
Fig, 5(a)



Fig, 5(b)



Fig, 6(a)



Fig, 6(b)

Fig. 6(b)

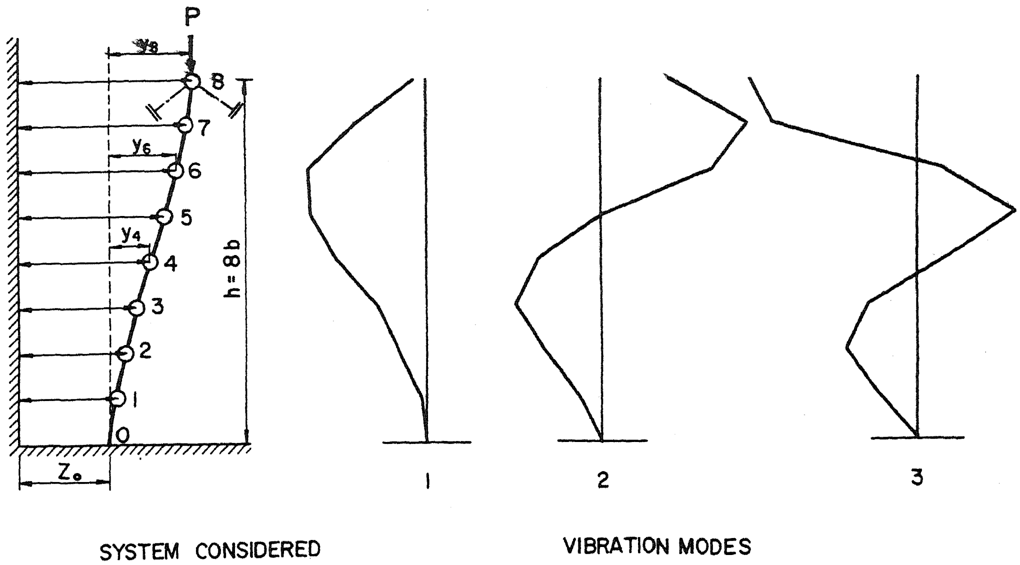


Fig. 7

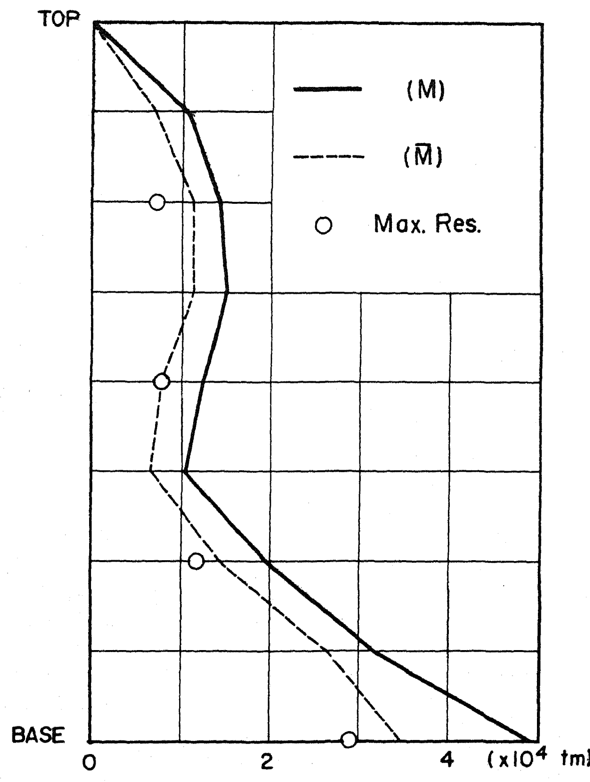


Fig. 8