SEISMIC ANALYSIS OF THE ELEVATED STRUCTURE FOR THE MEXICO CITY "METRO"

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Summary. Seismic analysis of the elevated structure for the Mexico City Metro is described. The structure was idealized as an inverted pendulum. Rotatory inertia and soil structure interaction effects were included in the dynamic analyses performed. A comparison with the results of the static analysis is made. Field tests to determine the actual dynamic properties in situ were carried out.

Introduction. An extension of the Metropolitan Transportation System (Metro) of Mexico City is under construction; it will have a new elevated line, 10 km long. Extensive studies were performed to determine the best type of structure, after which it was decided to use prestressed-concrete box-section beams, 8 m wide, cast in place and postensioned, with spans ranging from 25 to 40m supported on a single line of columns with variable cross section (fig. 1). The foundation consists of spread footings on friction piles.

Beam supports consist of neoprene and steel pads. Different thicknesses were used on each end in order to have a hinged-simple supported beam. Two pads on each side spaced 2.5m transversely to the beam take overturning effects. An extension of the end diaphragms enter a box left in the columns, to transmit all lateral loads to them. To avoid collapse of beams due to excessive movement during strong earthquakes tie-bars were used joining the ends of the two beams resting on each column.

Line loads are of two types: passenger trains with axle loads of 15.9 ton including impact, and a maintenance train, with axle loads of 25.0 ton. Different arrangements were used in order to obtain maximum effects when these loads were combined with earthquake.

Seismic analysis. The structure was analyzed using the Mexico City building code which specifies, for the high compressibility clay deposit where most of the line will be located, a seismic coefficient of 0.24 g, which should be increased 30 per cent for the case of special structures. To compute forces, this coefficient may be reduced according to ductility characteristics. For the Metro structure the reduction factor is 2; (ref.1).

The Code specifies that analyses may be static or dynamic. For the static analysis of inverted pendulum structures, defined as those having more than 50 per cent of the load concentrated at the top, with lateral forces resisted by a single element, rotatory inertia should be included using an expression given in the Code. An additional reduction of design forces is possible using a design spectrum and estimating the fundamental period of vibration. This reduction is generally possible in the case of very rigid structures on soft soil or flexible structures on stiff soil. Dynamic analysis may be step by step using four different accelerograms with intensity -

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compatible with the Code, or a modal analysis using a design spectrum.

To obtain seismic effects the Code specifies that the structure should be analyzed in two orthogonal directions. For the case of inverted pendulum structures, seismic effects in one direction and 50 per cent of the seismic effects on the other direction are combined with gravity loads.

Static Analysis. According to the Code, seismic effects for inverted pendulum structures consist of a horizontal force and a moment applied at the top. The horizontal force is equal to the mass times the seismic coefficient reduced by ductility. An increment of 30% was applied due to the importance of the structure. The moment at the top due to rotatory inertia should be computed as

$$M_o = 1.5\ V_o r_o^2\ \Theta_o/\delta_o$$

where V_o is the lateral force; r_o the radius of gyration of the mass with respect to a horizontal axis at the top of the structure, perpendicular to the direction of analysis; Θ_o, the rotation at the upper end due to V_o and δ_o the horizontal displacement of this point due also to V_o.

As it was mentioned before, additional reductions might be obtained in the case of rigid structures on soft soil, therefore, the fundamental period of vibration was estimated using the following expression, which is a modification of that proposed in the Code to take into account rotational effects:

$$T = 6.3\ \left[(m\delta_1^2 + J\Theta_1^2)/(V_o\delta_1 + M_o\Theta_1)\right]^{1/2}$$

Here δ_1 and Θ_1 are total displacements at the upper end due to the combined effect of V_o and M_o, m is the mass and J its polar moment of inertia.

Dynamic analysis. Three different models were considered for the dynamic analysis: cantilever column with mass concentrated at the top and perfectly fixed base, column with mass having rotatory inertia at the top and perfectly fixed base and column with mass having rotatory inertia at the top and soil-structure interaction at the base. Linear behavior was assumed, using the models proposed in ref.2.

For the first model the moment at the upper end is zero and frequency is equal to the square root of m over k. For the second case, the frequencies are given by

$$w_{1,2}^2 = \frac{kJ + mk_r}{2\ KmJ} \pm \left(\frac{kJ + mk_r}{2KmJ} - \frac{k\ k_r - \upsilon^2}{K^2\ mJ}\right)^{1/2}$$

where k is translational stiffness; k_r rotational stiffness; $K = 1-\delta\theta$; δ is the horizontal displacement at the top due to a moment k_r; Θ is the rotation at the top due to a horizontal force k; $\upsilon=\gamma k\cdot k_r$, γ is the rotation at the top due to a unit horizontal load or the lateral deformation due to a unit moment applied at the top.

Table 1 summarizes the above elastic properties of the column for both directions of analysis; table 2 shows values of m and J corresponding to the most adverse arrangement of live load.

Modal configurations for the second case are given by

$$x_{ij}/\varepsilon_{ij} = {}^{k\delta}/K({}^{k}/K-mw_j^2)$$

where x and ε are total displacements and rotations.

The spectrum used corresponds to the soft soil of the city and is described by a = (0.06 + 0.225 T) g for T <0.8 sec; a = 0.24 g for 0.8 sec <T<3.3 sec; a = (0.792/T) g for T >3.3 sec. Where a, the spectral acceleration, may be reduced to compute seismic forces dividing by a ductility reduction factor Q = 2 for T>0.8 sec or by Q' = 1 +1.25 T for T<0.8 sec.

The fundamental period of the structure is smaller than 0.8 sec, therefore any increase in its value due to soil-structure interaction would increase the response and model 3 was necessary. As the structure is supported on friction piles the dynamic properties of the foundation are difficult to evaluate. As mentioned before the model used is described in ref. 2, it does not include the mass of foundation and adjacent soil. Stiffnesses in translation and rotation of the group of piles were computed using Hrennikof's method (ref. 3) Lateral stiffness computed is 21 000 ton/cm and rotational stiffness 3 200 000 ton-m/rad. The mechanical elements obtained by the three dynamic models are presented in table 3 for the least favorable load combination.

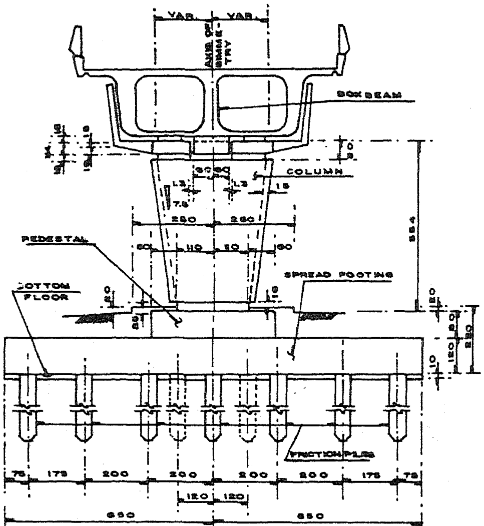
Comparison of results. It may be observed in table 3 that the moment computed by the static method is larger than that obtained with dynamic models 1 or 2, however, lateral force is larger in the dynamic model with soil-structure interaction and the moments at the base are larger than those computed statically.

Combination of effects in both directions leads to similar results in the static and dynamic analyses.

Research program and field tests. In order to evaluate the dynamic parameters of the structure a research project is underway at the writing of this paper. It includes free and forced vibration of beams and columns to measure effective modulus of elasticity, periods of vibration and soil-structure interaction effects, as well as theoretical studies to analyze more sophisticated models of soil-structure interaction and step by step analyses with typical accelerograms recorded on the soft-soil of Mexico City. Fig 2 shows a general view of the tests. Due to space limitations results of this research program may be described during the conference.

References.

1. Mexico City building code, 1976
2. Rascón, O.A. "Seismic effects on inverted pendulum structures" (in spanish). Rev. Soc. Mex. Ing. Sism., 1965
3. Bowles, J. Foundation Analysis and Design, McGraw Hill Book, Co., 1968.



F I G. 1

TABLE 1

COLUMN PROPERTIES					
X DIRECTION			Z DIRECTION		
k	73214.37	ton./m	k	70684.7	ton./m
kr	1149521.35	ton.m/rad	kr	892233.	ton.m/rad
δ	3.158x10-6	1/ton.	δ	3.515x10-6	1/ton.
θ	0.2311	rad./m	θ	0.24850	rad / m
ξ	3.63	m/rad.	ξ	3.14	m /rad.
K	0.160		K	0.220	
γ	265760.47	ton.	γ	221723.	ton.

TABLE 3

COMPARATIVE RESULTS						
ANALYZED MODEL		EARTHQUAKE DIRECTION	FORCES AND BENDING MOMENTS AT BASE OF COLUMN			
			V (ton.)	%	M (ton.-m)	%
STATIC ANALYSIS		X	126.27	100.0	1231.13	100.0
		Z	116.56	100.0	667.60	100.0
DYNAMIC ANALYSIS		X	117.91	93.4	641.44	52.1
		Z	115.82	99.4	630.06	94.4
		X	81.86	64.8	719.36	58 4
		Z	111.12	95.3	631.17	94.5
		X	138.56	109.7	926.16	75.2
		Z	143.71	123.3	791.83	118.6

TABLE 2

m AND J FOR ONE LOADING CONDITION							
EARTHQ. DIR.	MASS (ton.-seg^2/ m)				ROTATORY INERTIA (ton.-m-seg^2)		
	GIRDER+ ADD.LOAD	COLUMN	LIVE LOAD	TOTAL	GIRDER+ ADD.LOAD	LIVE LOAD	TOTAL
X	76.45	7.01	22.11	109.58	745.41	566.83	1312.24
Z	76.45	7.01	20.04	103.51	61.92	17.91	79.83

FIG. 2