DYNAMIC ANALYSIS OF A 250 m HIGH REINFORCED CONCRETE CHIMNEY AND CRITERIA FOR ITS SEISMIC STABILITY

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This paper reviews the design development of a 250 m high reinforced concrete free standing chimney recently constructed in Bitola, Yugoslavia. Included is a description of the structural configuration and criteria by which it was designed. The dynamic characteristics of a linear elastic mathematical model to represent the physical structure are presented. Also, included in the comparison is the incluence of the soil-structure interaction on the dynamic characteristics of the chimney. The dynamic response of this model to various levels of ground shaking is evaluated, and compared to the seismic force levels prescribed by the 1964 edition of the Yugoslav Building Code. The wind action is the basic loading for this kind of structures, therefore, the wind effect upon the chimney is presented and the wind design criteria are given.

INTRODUCTION

The present enviroment protection codes have been requiring lately that the structures like chimneys of industrial plants and energy producing facilities be constructed higher than the technological process requires. Thus, the problems concerning determination of loads due to wind and seismic ground motion action, especialy in the case of extraordinary high action values prove to be of the highest importance for rational design. The problem of aseismic design of these structure is in principle not different from the one refered to other civil engineering structures, since fundamentally it is the same. The present world experience on the behaviour of these structures under strong earthquakes is mimimum. The existing construction regulation in the world for definition of design criteria of these structures also differs. Generally considered, construction of the structures is based upon traditional experience, therefore it is considered to be of interest to present such an example.

STRUCTURAL CONFIGURATION AND FOUNDATION

The geometrical profile of the chimney body consists of three inclined lines of its external face of the chimney: the first section of 55 m - i_1= 4,5%, the second one of 120 m - i_2 = 1,92%, and the third one is vertical, i.e. i_3 = 0%. The size of the reinforced concrete profile changes linearly along the hight of teh chimney. Fig. 1a illustrates the longitudinal cross section with the basic proportions of the chimney, where D is the external diameter and d the thickness of the ring for the characteristic cross sections. The foundation is constructed as a slab.

MODELLING

Selection of the mathematical model is the initial and basic problem in structural analysis for given actions. The problem is directly related to the structure as structural model represents idealization of the actual structure. The following basic assumptions have been taken for definition of the mathematical model: the system is linear; it is considered as discrete; there are classical, normal modes of vibration of the model; all nodes of the model has two degrees of freedom (horizontal translation and rotation around the z axis); rotational, inertial forces have been neglected. Presented in Fig. 1b is the adopted model, which represents a linear system with beam member considered as a basic member of the model. The effective masses are distributed at the points. Soil structure interaction is modelled by two elastic springs simulating rotation and translation of the foundation. The elastic spring characteristics are determined from the geometry of the foundation structure and soil mechanical characteristics(1).

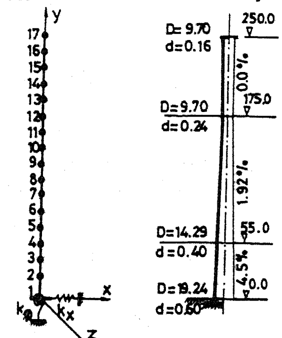

Fig. 1

DYNAMIC CHARACTERISTICS

For illustration, the calculated undamped natural periods of vibration for the first five basic modes of vibration for the model are presented:

	T_1(sec)	T_2(sec)	T_3(sec)	T_4(sec)	T_5(sec)
Model "A"	4.998	1.238	0.505	0.272	0.160
Model "B"	5.582	1.446	0.592	0.307	0.185

The "B" model simulates rotation of the foundation structure (for soil characteristics $G=6.000\ t/m^2$ and $\nu=0.3$) which is not the case with the "A" model.

DESIGN LOADS

Seismic loads, as specified by the 1964 edition of the Yugoslav Building Code and those resulting from a linear elastic dynamic analysis of the idealized model of ten chimney structures when subjected to the recommended two levels and two types of ground shaking. The unfactored El Centro May 1940 component record, and the unfactored Parkfield 227/6/66 N65E motions records were used. The unfactored records used to represent ground excitations were identified with two levels values: the design earthquake with $\alpha p = 0.165$ g and the maximum probability earthquake $\alpha_{max} = 0.225$ g. Constant, 5% of critical viscous damping was assumed in all analysed mode vibration.

Wind loads, as specified by the DIN Code except as noted in the equation $Q(H) = 120 + 0.6H$, when Q (kg/m^2), $H(m)$.

CRITERIA FOR SEISMIC STABILITY

Under the combined action of dead loads and simultaneously applied ground excitations, as defined in the previous section, all the sections of the chimney structure shall be reasonably expected to survive, without collapse, if the factors of safety are more than one for the design earthquakes and the maximum probability earthquakes. The safety coefficients are defined as for the design value it is $k_1 = M^Y / M^{Sdesign}$

and for maximum possible values $k_2 = M^u/M^{Smax}$

Determination of yield point of elasticity

It is shown in Fig. 2 the accepted diagram of the stress distribution in the ring cross section with middle radius r and thickness δ at the moment of reinforcement yielding due to the effect of the eccentric force N. A linear function of stress distribution have been accepted.

The angle α defines the position of the neutral axis at the moment of yielding of reinforcement. The angles β and γ define the cross sections in the tensile i.e. compressed zone. If yielding of reinforcement is identified with σ_a^Y, then all the remaining stresses in the individual profiles of the concrete and the reinforcement can be expressed as a function of σ_a^Y :

$$\sigma_{\gamma a} = \frac{\cos\gamma - \cos\alpha}{1+\cos\alpha} \cdot \sigma_a^Y \qquad \sigma_{\gamma b} = \frac{\sigma_{\gamma a}}{n} = \frac{\cos\gamma - \cos\alpha}{1+\cos\alpha} \cdot \frac{\sigma_a^Y}{n} \qquad \sigma_{\beta a} = \frac{\cos\alpha - \cos\beta}{1+\cos\alpha} \cdot \sigma_a^Y \qquad n = \frac{E_a}{E_b}$$

The equilibrium condition in the cross section define two equations:

$$\Sigma P = 0 \quad N + V = C \qquad \Sigma M = 0 \quad N e_o = M_C + M_1$$

Representing the internal forces in fuction of the reinforcement percentage μ, the parametre α which defines the position of the neutral axis x, and yield stresses in the reinforcement σ_a^Y, the equation from which the parametre α is determined can be obtained:

$$\frac{\sigma_o}{\sigma_a^Y} = \frac{1}{n\pi} \frac{\sin\alpha - \alpha\cos\alpha - n\pi\mu\cos\alpha}{1 + \cos\alpha}$$

The second equation can be written as: $M^Y = \dfrac{A_b\, r\, \sigma_a^Y(\alpha - \sin\alpha \cos\alpha + n\pi\mu)}{2\pi\, n \quad (1 + \cos\alpha)}$

The expression from which the stresses in the concrete at the moment of reinforcement yielding are obtained is : $\sigma_b' = \dfrac{\sigma_{ac}}{n} = \left(\dfrac{1-\cos\alpha}{1+\cos\alpha}\right) \dfrac{\sigma_a^Y}{n}$

where: α - angle defining the neutral axis, M^Y - yield point of elasticity; σ_o - centric pressure; A_b - area of concrete; r - middle radius of the ring; σ_a^Y - yield point of steel; μ - percentage of reinforcement.

Determination of ultimate bearing capacity

The fig. 3 shows the diagram of the stress state in the ring cross section with middle radius r and thickness of the ring δ at the moment of collapse due to eccentric force N. Rectangular distribution of stresses in the compressed and tensile zone have been accepted. The angle θ defines the position of the neutral axis, while ʟ is the area of the compressed part.

Applying the equilibrium conditions it is obtained:

$$\Sigma P=0 \quad N+V_{a1}+V_{a2}=C_b+C_a \qquad \Sigma M=0 \quad N\, e_o=C_b\, Z+2\, C_a\, Z$$

$$\text{where, } C_b=\frac{\delta\pi r R i}{90^o}\,\theta \qquad C_a=\frac{\mu A b\sigma_a^Y}{180^o}\,\theta \qquad V_{a2}=\mu A b\sigma_a^Y\left(1-\frac{\theta}{90^o}\right) \qquad V_{a1}=C_a$$

The equilibrium equations can be written in the following form:

$$\theta^o=180^o\,\frac{\sigma_o/R_i+\mu\,\sigma_a^Y/R_i}{1+2\mu\,\sigma_a^Y/R_i} \qquad M_o^u=\frac{Ab}{\pi}\,2(R_i+2\mu\sigma_a^Y)\,\sin\theta$$

The first equation defines the neutral axis and the second one defines ultimate bearing capacity, where R_i - stresses of concrete at the moment of collapse

In this case, the ultimate moment M^u is defined as a stress condition in the ring section at the moment when the maximum compressed part reaches the value of σ_b^y = 0.85 Ri. That means that the reinforcement is yielding, but not the concrete which is a more rigorous criterion of security than in the case of M^u_o. In Fig.4 are presented the diagrams of the computed values of M_i^y, M_i^u and M_{oi}^u (see Fig 4b) for the determined sections of the mathematical model with σ_a^y = 4000kp/cm^2, Ri = 300kp/cm^2, n = 15m and a corresponding procentage of reinforcement of section μ (see Fig. 4a). The diagrams of the maximum concrete and steel stresses under the influence of the designed seismic forces and the wind are also presented in Fig. 4c.

CONCLUSION

On the basis of the already done analysis and accepted security criterion, the following can be concluded:

- For the influence of the designed seismic forces, the analysed sections of the reinforced concrete structure are in a stress condition with a security coefficient close to the reinforcement yield point which has an average value of 1.20 and higher value for the lowest and the highest sections (Fig. 5a).
- For the influence of maximum seismic forces, the analysed sections of the reinforced concrete structure are in a stress condition with a security coefficient of 1.5 - 1.1, the former value refering to the lower sections, the letter refering to the upper sections while for the higher sections the coefficient is higher than 2.
- The comparison between the moments of the ultimate bearing capacity of the analysed sections and the moments of the application of force obtained by the influence of the maximum seismic forces, shows that all of the sections of the designed reinforced concrete structure are in an elastic condition of stress σ_a < 4000kg/cm^2 with the exception of the sections which are 145 to 190m high and are in a condition of a yielding reinforcement (see Fig. 5c).

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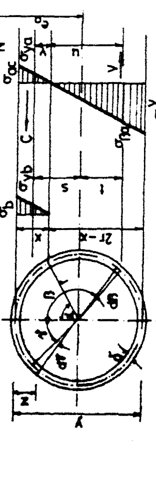

Fig. 2

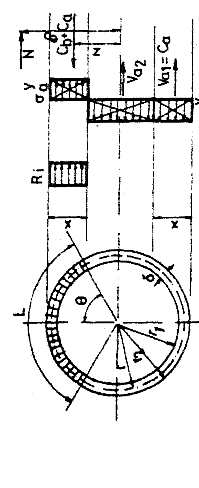

Fig. 3

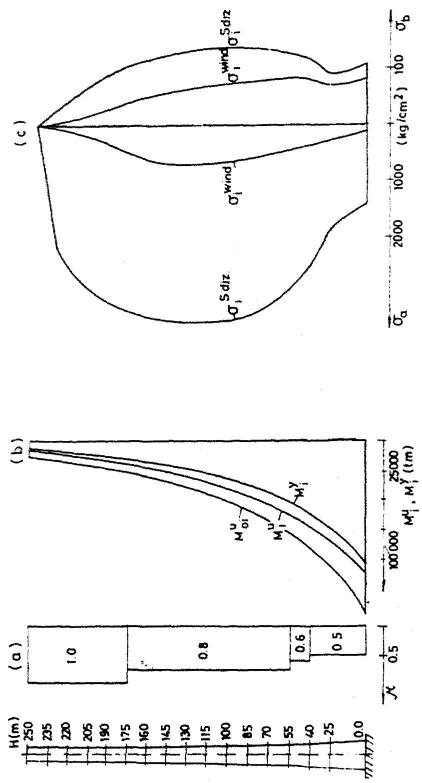

Fig 4

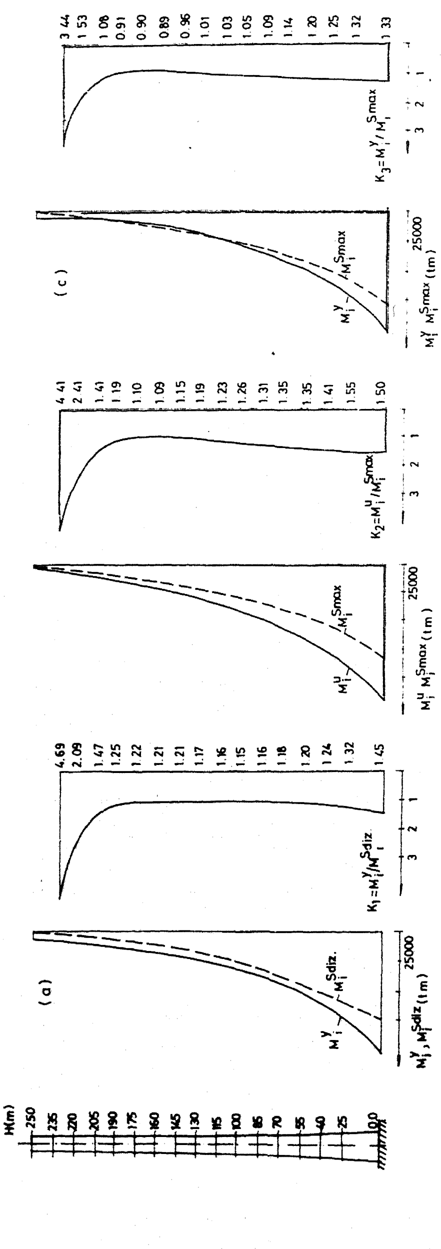

Fig 5