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Seismic design of buildings with multi-level basements

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ABSTRACT: A simplified, approximate procedure is proposed for the seismic design of high-rise buildings with multi-level basements, which takes into account the superstructure-substructure-soil interaction. In addition, dynamic earth pressures developed on the exterior walls of the substructure are considered.

1 INTRODUCTION

In current (1991) practice of seismic design of high-rise buildings with multi-level basements (Figure 1a) it is generally assumed that the building is fixed-supported at the ground level, and the superstructure (Figure 1b) is designed accordingly. However, the basement substructure (Figure 1c) does not necessarily provide a fixed support, and the superstructure may experience both a horizontal translation, u, and a rotation (rocking), ϕ, at the ground level. Assuming that the substructure is rigid, it experiences the same displacements, u and ϕ. These forced displacements, in turn, result in dynamic earth pressures on the basement exterior walls, which should also be taken into account in seismic design.

A simplified, approximate procedure is proposed to take into account the superstructure-substructure-soil interaction in an idealized manner. The procedure follows the guidelines proposed by the U.S. National Earthquake Hazard Reduction Program, NEHRP (1988). In addition, a simplified, approximate approach is presented to estimate dynamic earth pressures on the basement walls.

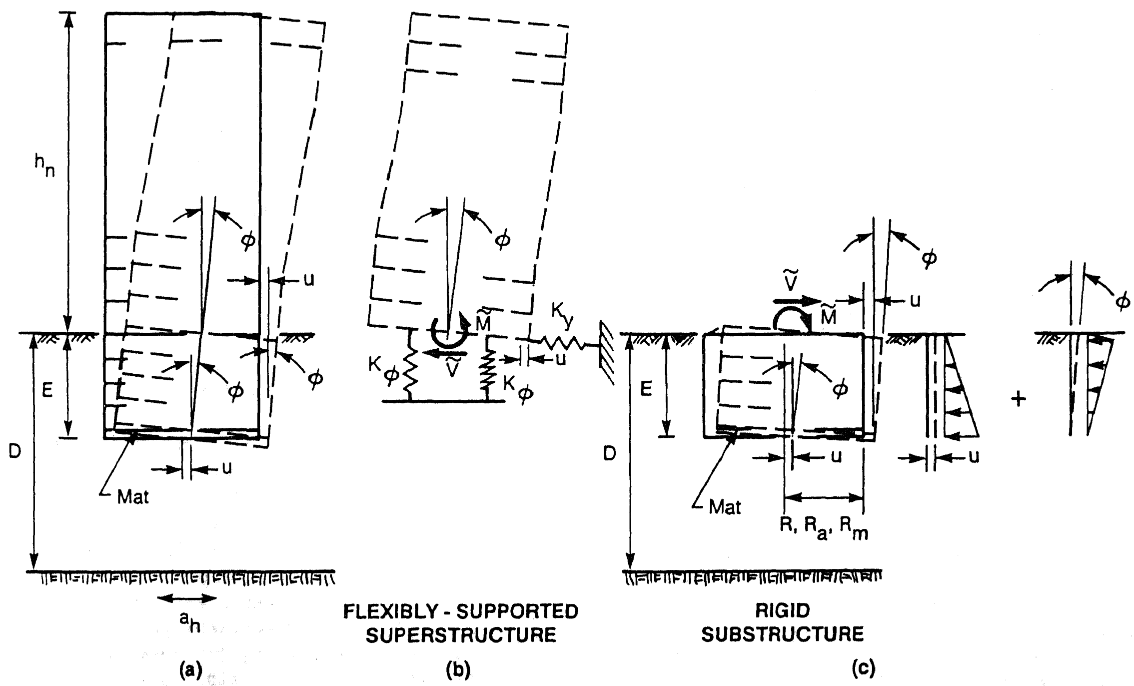

Figure 1. High-rise building with multi-level basement: Interactive seismic design approach.

2 SEISMIC DESIGN OF SUPERSTRUCTURE

The base shear is calculated assuming that the superstructure is fixed at the ground level. The calculated base shear, V, is then reduced by ΔV due to soil-structure interaction.

$$V = C_s W \tag{1}$$

$$C_s = (1.2\ A_v S/RT^{2/3}) \tag{2}$$

where C_s is seismic design coefficient, W is total gravity load from the superstructure, A_v is effective peak-velocity related acceleration coefficient which is provided for each county in U.S., S is soil (site) coefficient, R is response modification factor for the fixed-base superstructure, and T is fundamental natural period of the fixed-base superstructure.

$$\Delta V = [\tilde{C}_s - C_s\ (0.05/\tilde{B})^{0.4}]\bar{W} \tag{3}$$

where $\tilde{C}_s$ is seismic design coefficient for the flexibly-supported superstructure with fundamental natural period, $\tilde{T}$, and it is calculated from Equation 2. $\tilde{B}$ is fraction of critical damping for the structure-soil system, and $\bar{W}$, effective gravity load of the superstructure taken as 0.7 W.

Fundamental natural period of the flexibly-supported structure is calculated from:

$$\tilde{T} = T[1+(\bar{k}/K_y)(1+K_y\bar{h}^2/K_\phi)^{1/2}] \tag{4}$$

where $\bar{k}$ is stiffness of the fixed-base superstructure, $\bar{k} = (4\pi^2\bar{W})/(gT^2)$, $\bar{h}$ is effective height of the superstructure taken as $0.7h_n$, and g is acceleration of gravity. K_y is lateral stiffness of the substructure-soil system, and it is defined as the static horizontal force applied at the level of the foundation mat which is necessary to produce a unit lateral deflection of the substructure at that level. Similarly, K_ϕ is rocking stiffness of the substructure-soil system, and it is defined as the static moment applied about the axis through the centroid of the foundation mat which is necessary to produce a unit rotation of the substructure (Figure 1c).

Elsabee (1975), Elsabee et al (1977), NEHRP (1988) and Gazetas (1991) proposed:

$$K_y = [(8GR)/(2-\nu)](1+R/2D)(1+2E/3R)(2+5E/4D) \tag{5}$$

$$K_\phi = [(8GR^3)/3(1-\nu)](1+2E/R)(1+R/6D)(1+0.7E/D) \tag{6}$$

where G is average shear modulus of the soil at "large" strains, surrounding and underlying the substructure, is average Poisson's ratio of the soil, E is depth of embedment of the substructure, D is thickness of the total soil stratum, and R is the radius of the cylindrical substructure (see Figure 1). For a rectangular (prismatic) substructure equivalent radii, R_a and R_m, to be used in Equations 5 and 6, respectively, are $R_a = (A_o/\pi)^{1/2}$ and $R_m = (4I_o/\pi)^{1/4}$, where A_o is the area and I_o is the moment of inertia of the rectangular base in the respective orientation. Equations 5 and 6 are applicable for the range of $E \leq 0.5D$ and $R \leq 0.5D$.

$$\tilde{B} = B_o + 0.05/(\tilde{T}/T)^3 \tag{7}$$

where B_o, the foundation damping factor, incorporates both radiation and material damping of the soil, and is provided in NEHRP (1988) in term of system geometry and period. The second term in Equation 7 represents structural damping.

With V and ΔV calculated from Equations 1 and 3, respectively, the reduced base shear for the flexibly-supported superstructure, $\tilde{V}$, is:

$$\tilde{V} = V - \Delta V \tag{8}$$

NEHRP (1988) specifies that $\tilde{V}$ shall in no case be taken less than 0.7 V.

3 SEISMIC DESIGN OF SUBSTRUCTURE

The seismic design of the substructure shall take into account $\tilde{V}$, and the associated moment, $\tilde{M}$, induced by the flexibly-supported superstructure. In addition, dynamic earth pressures, associated with the forced displacement of the rigid substructure, u and ϕ, shall be included in the design.

For the rigid substructure Elsabee et al (1977) formulated the force-displacement relations as:

$$P = K_y u + K_{y\phi}\phi \tag{9}$$

$$M = K_{\phi y} u + K_\phi \phi \tag{10}$$

where K_y and K_ϕ are provided by Equations 5 and 6, respectively. $P = \tilde{V}$ is the lateral force applied at the centroid; and $M = \tilde{M} + \tilde{V}E$ is the moment about the axis through the centroid of the foundation mat (Figure 1c). The coupled spring constant was proposed as:

$$K_{\phi y} = K_{y\phi} = K_y R[(2E/5R) - 0.03] \tag{11}$$

Thus, forced displacements, u and ϕ, may be determined by solving Equations 9 and 10 simultaneously.

For the calculated u and ϕ values, the corresponding dynamic earth pressures may be estimated by using the results of the analytical work of Wood (1986) undertaken within the comprehensive research program at the Central laboratories, New Zealand, to study the seismically induced earth pressures against monolithic (rigid) bridge abutment walls. The results of Wood's (1986) analytical work were generally confirmed by the complimentary model wall tests reported by Thurston (1986a, 1986b, 1987).

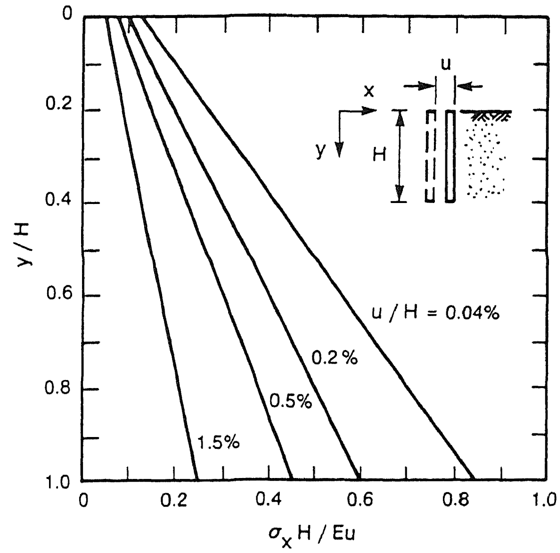

Figure 2. Dimensionless pressure vs. height for rigid wall translated into soil/backfill.

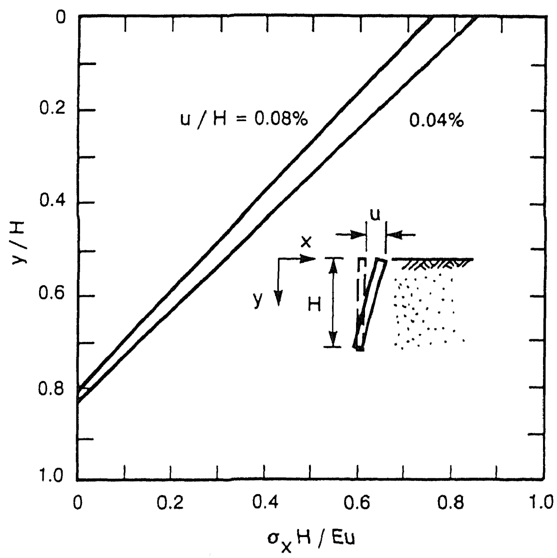

Figure 3. Dimensionless pressure vs. height for rigid wall rotated into soil/backfill.

In Figure 2 dimensionless wall pressure distributions for a rigid wall translated against a dense sand backfill are presented for a range of dimensionless wall translations. The figure was prepared by the authors as a composite of Wood's (1986) analytical solutions, and Thurston's (1986a, 1987) model test results. Similarly, in Figure 3 dimensionless wall pressure distributions for a rigid wall rotated about its base against a dense sand backfill are presented for the range of indicated wall rotations. Figure 3 was also prepared by the authors as a composite of Wood's (1986) analytical solutions and Thurston's (1986b) model test results. The resultant wall pressures due to forced horizontal translation, u, and rotation, ϕ, are the sum of the two components. The pre-earthquake static at-rest earth pressures are also included in Figures 2 and 3.

4 SOIL PARAMETERS FOR DESIGN

The soil parameters utilized in the interaction analysis presented are the shear modulus or the associated shear wave velocity, the soil unit weight, and the Poisson's ratio. These properties generally vary spatially, and it is necessary to use average values for the soil zone that is affected by the forces acting on the substructure. The depth of influence is a function of the geometry of the substructure and the mode of motion involved. NEHRP (1988) suggests that the affected soil zone would extend to $4R_a$ or $1.5R_m$ below the foundation mat for horizontal translation and rocking, respectively.

The shear modulus, G, in Equations 5 and 6 shall be interpreted as the secant modulus corresponding to the strain level which increases with the intensity of the ground shaking. NEHRP (1988) provides an approximate correlation between G and the "small strain" shear modulus, G_o, as a function of A_v (Equation 2). G_o may be most conveniently determined from in-situ shear wave velocity, v_{s0}, measurements within the zone of influence. NEHRP (1988) also provides empirical correlations to obtain approximate v_{s0} values from index properties of the soil layers in the affected zone.

As for the modulus of elasticity, E, of the backfill soils in Figures 2 and 3, Wood's (1986) results were formulated in terms of the average "small strain" modulus, which is associated with G_o, within the depth of the wall. As indicated E or G at small strains may be obtained either by in-situ shear wave velocity measurements or laboratory undrained triaxial-compression tests and determining the initial tangent moduli.

It would be satisfactory to use a Poisson's ratio of 0.33 for sands and gravels, 0.40 for stiff clays, and 0.45 for soft clays (NEHRP 1988).

5 MEASURED PERFORMANCE DATA

During the 1989 Loma Prieta earthquake, M_s = 7.1, performance of the 60-story, pyramid-shaped Transamerica Building (Figure 4) in San Francisco was recorded by a network of instruments (Celebi and Safak 1991). Within the context of the paper acceleration records from two of the instruments located in the substructure, and designated as 10 and SMA are referenced (Figure 4).

Processed acceleration records from uniaxial accelerometer 10 and strong motion accelerograph SMA indicated that at 10.7 sec. of the ground shaking,

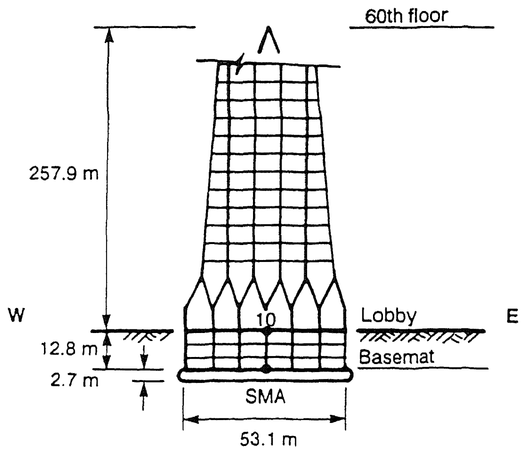

Figure 4. Transamerica building, San Francisco and referenced instrumentation (Celebi & Safak 1991).

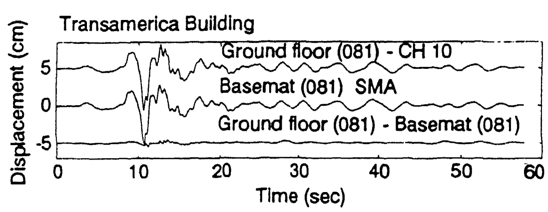

Figure 5. Horizontal translation computed from acceleration records of instruments No. 10 and SMA.

a maximum horizontal translation of 5.62 cm. and 5.24 cm. occurred at the lobby level (10) and the basemat (SMA), respectively (Figure 5). For a rigid assumed substructure this corresponds to a rigid-body translation of 5.24 cm. to the east, and a rigid-body rotation of 0.017 degrees about the N-S axis through the centroid of the basemat. Regarding the east wall of the substructure, this corresponds to a 0.4% translation, u/H (Figure 2), and a 0.03% rotation, ϕ (Figure 3).

Celebi and Safak (1991) reported that the peak differential horizontal displacement measured by instruments 10 and SMA (Figure 4) was 0.59 cm at approximately 11 seconds into the record. This corresponds to a 0.046% maximum rotation (rocking) about the N-S axis through the centroid of the basemat.

6 CONCLUSIONS

It is suggested that in seismic design analyses of buildings with multi-level basements the superstructure-substructure-soil interaction be taken into account as a more rational approach than the currently practiced fixed-base superstructure assumption. This would also provide a more economical design in general.

A simplified, approximate procedure utilizing interaction springs is outlined for representation of the superstructure-substructure-soil interaction following and extending the NEHRP (1988) guidelines. The procedure sheds more light to the seismic design of the substructure which is often overlooked. Dynamic earth pressures developed as a result of the forced translation and rotation (rocking) of the substructure are also taken into account in the proposed procedure.

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