Seismic design of earth-retaining structures and foundations

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Earthquake-resistant design of earth retaining structures like retaining walls, earth dams and foundations are very important problems to minimize the devastating effect of earthquake hazards. In this paper a comprehensive review for different methods to calculate seismic earth pressures and their point of applications is shown. Numerical example shows the merit of a displacement-based analysis over force-based analysis by considering a permissible displacement of the wall. Modification of IS code for seismic design of retaining wall is proposed. Merits and demerits of several design and analytical procedures for earthen dam and foundations under seismic conditions are provided in this paper.

DESIGNS of retaining walls, earth dam, abutments, foundations, etc. are very important problems related to the geotechnical engineering. To minimize the devastating effect of earthquake on retaining structures, computation of earth pressures and the point of applications carries attention of the researchers. The most commonly used methods to design the retaining structures under seismic conditions are the force equilibrium based pseudo-static analysis (for e.g. refs 1–7), pseudo-dynamic analysis (ref. 8), displacement based sliding block methods (for e.g. refs 9–12). In the design of these retaining structures, knowledge of earth pressures under both active and passive conditions should be cleared to compute the earth forces and their point of applications. Design of earth dam under seismic condition is a challenge to the geotechnical engineers. Slope stability method is basically used for the analysis of earth dam using pseudo-static approach for seismic forces. Shallow foundations are also designed under earthquake condition by computing the bearing capacity using pseudo-static methods available in literature. This paper reviews the different methodologies available to compute the active and passive earth pressures under seismic conditions for retaining wall, design of earth dam and shallow foundations and the mitigation techniques to the earthquake hazards are also discussed.

Seismic earth pressures on retaining wall

In the design of retaining wall, the earth pressure (active, passive or at-rest, depending on the wall movement) has to be computed properly. Broadly, force-based and displacement based analysis are used to compute these seismic earth pressures.

Force-based analysis

The seismic stability of earth-retaining structures is usually analysed by the pseudo-static approach in which the effects of earthquake forces are expressed by constant horizontal and vertical accelerations attached to the inertia. The common form of pseudo-static analyses considers the effects of earthquake shaking by pseudo-static accelerations that produce inertial forces, F_h and F_v, which act through the centroid of the failure mass in the horizontal and vertical directions respectively. The magnitudes of the pseudo-static forces are,

$$F_h = \frac{a_h W}{g} = k_h W, \tag{1}$$

$$F_v = \frac{a_v W}{g} = k_v W, \tag{2}$$

where a_h and a_v are the horizontal and vertical pseudo-static accelerations, k_h and k_v are coefficients of horizontal and vertical pseudo-static accelerations and W is the weight of the failure mass.

Guttenberg and Richter (see ref. 6) proposed the following equation to calculate maximum seismic acceleration a_0

$$\log_{10} a_0 = -2.1 + 0.81M - 0.027M^2, \tag{3}$$

where M is the Richter scale magnitude of earthquake for epicentral region and a_0 is the maximum seismic acceleration, $k_h g$ (cm/s²).

A pseudo-static analysis is relatively simple and very straightforward. Representation of the complex, transient, dynamic effects of earthquake shaking by a single constant unidirectional pseudo-static acceleration is obviously quite crude. However, experiences have shown that pseudo-static analyses can be unreliable for soils that build up large pore pressures or show more than about 15% degradation of strength due to earthquake shaking. Okabe[2], Mono-

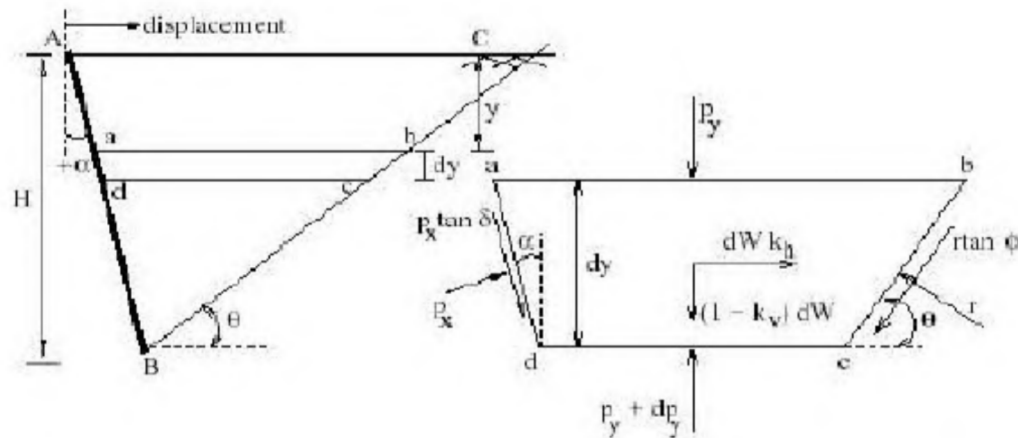*For correspondence. (e-mail: dc@civil.iitb.ac.in)

nobe and Matsuo[1] (see ref. 13) were the early pioneers to obtain the active and passive earth pressure coefficients under seismic conditions. It was an extension of Coulomb's method in the static case for determining the earth pressures by considering the equilibrium of triangular failure wedge. The method is now commonly known as Mononobe–Okabe method. Here to compute the active and passive earth pressure by pseudo-static forces as the seismic forces acting on the backfill cohesionless soil, planar rupture surface was assumed in the analysis. But one important criterion of the point of application of the seismic earth pressure cannot be obtained from this method. The seismic active and passive earth pressure (P_{ae}, P_{pe}) can be computed by Mononobe–Okabe equation as follows

$$P_{ae,pe} = \frac{1}{2}\gamma H^2 (1 - k_v) K_{ae,pe},$$ (4)

and

$$K_{ae,pe} =$$

$$\frac{\cos^2(\phi \mp \beta - \theta)}{\cos\theta \cos^2\beta \cos(\delta \pm \beta + \theta)\left[1 - \left(\frac{\sin(\phi + \delta)\sin(\phi \mp i - \theta)}{\cos(\delta \pm \beta + \theta)\cos(i - \beta)}\right)^{0.5}\right]^2},$$ (5)

where γ = unit weight of soil; H = vertical height of the wall; $K_{ae,pe}$ = seismic active and passive earth pressure coefficient; ϕ = soil friction angle; δ = wall friction angle; β = wall inclination with respect to vertical; i = ground inclination with respect to horizontal; k_h = seismic acceleration coefficient in the horizontal direction; k_v = seismic acceleration coefficient in the vertical direction.

$$\theta = \tan^{-1}\left[\frac{k_h}{1 - k_v}\right].$$

Though Mononobe–Okabe analysis assumes that the point of application of the total seismic earth force is at $H/3$ from base of the wall, experimental results show it should act above $H/3$ from base of the wall for seismic active case. Seed and Whitman (see ref. 13) recommended that the dynamic component be taken to act at $0.6H$. Also, Terzaghi[14] had shown that for active earth pressure, assumption of planar rupture surface in the analysis closely matches with the actual experimental observations. But for passive case, when wall friction angle, δ exceeds one-third of soil friction angle, ϕ (i.e. $\delta > \phi$) then the assumption of planar failure surface seriously overestimate the passive earth pressures. Hence to correct the error in Mononobe–Okabe method for passive case, Morisson and Ebeling[3], Soubra[4] and Kumar[5] had considered curved rupture surface in their analysis for passive case. Recently Choudhury[6] had given the complete solution for passive earth pressure coefficients under seismic conditions for variations in parameters, like wall inclination, ground inclination, wall friction angle, soil friction angle and wall adhesion for a generalized $c - \phi$ (where c is the soil cohesion) backfill. Figure 1 shows the forces and failure surface considered in the analysis.

The points of applications for total passive resistance under seismic condition were determined by Choudhury et al.[15] by using method of horizontal slices. Figure 2 shows the analytical model considered for the analysis. The results showed that the values of points of application of total passive resistance ranges from $0.28H$ to $0.4H$

Figure 1. Failure surface and forces for seismic passive resistance computation[6].

Figure 2. Analytical model for obtaining the point of application of seismic passive resistance[15].

from base of the wall with vertical height H, for different seismic conditions and wall inclinations. One such typical result is shown in Figure 3 where the variation of seismic passive earth pressure with respect to depth is shown for different wall inclinations (α), soil friction angles (ϕ), wall friction angles (δ), horizontal seismic acceleration coefficient (k_h) and vertical seismic acceleration coefficient (k_v). From Figure 3, it is clear that the point of application of total passive resistance need not act always at $0.33H$ from the base of the wall with vertical height H, but it can vary. Hence the distribution of seismic passive earth pressure along the depth is nonlinear in most of the cases.

In the pseudo-dynamic approach, the advancement over the pseudo-static approach is that the dynamic nature of the earthquake loading is considered in an approximate but simple manner. The phase difference and the amplification effects within the soil mass are considered along with the accelerations to the inertia. Steedman and Zeng[8] considered the harmonic horizontal acceleration of amplitude a_h at base of the wall, then at depth z below the ground surface the acceleration can be expressed as

$$A(z, t) = a_h \sin[\omega\{t - (H - z)/v_s\}], \tag{6}$$

where ω = angular frequency; t = time elapsed; v_s = shear wave velocity.

The planer rupture surface inclined at an angle α with horizontal is considered in the analysis along with the seismic force and weight of failure block. The total seismic active force (P_{ae}) on the wall is given by,

$$P_{ae}(t) = \frac{Q_h(t)\cos(\alpha - \phi) + W\sin(\alpha - \phi)}{\cos(\delta + \phi - \alpha)}, \tag{7}$$

and

$$Q_h(t) = \frac{\lambda \gamma a_h}{4\pi^2 g \tan\alpha}[2\pi H \cos\omega\zeta + \lambda(\sin\omega\zeta - \sin\omega t)], \tag{8}$$

where $\lambda = 2\pi v_s/\omega$; $\zeta = 1 - H/v_s$.

The point of application of the total seismic active force is h_d from the base of the wall and given by

$$h_d = H - \frac{2\pi^2 H^2 \cos\omega\zeta + 2\pi\lambda H \sin\omega\zeta - \lambda^2(\cos\omega\zeta - \cos\omega t)}{2\pi H \cos\omega\zeta + \pi\lambda(\sin\omega\zeta - \sin\omega t)}. \tag{9}$$

This point of application of the seismic force for very low frequency motions (small H/λ, so the backfill moves essentially in phase) is at $h_d = H/3$. For higher frequency motions, h_d moves upwards from base of the wall.

Displacement-based analysis

A retaining structure subjected to earthquake motion will vibrate with the backfill soil and the wall can easily move from the original position after the earthquake. The methods available for displacement-based analysis of retaining structures during seismic conditions are based on the early work of Newmark[16]. The basic procedure for evaluating the deformation of an embankment dam shaken by earthquake was by considering sliding block-on-a-plane mode. Newmark's model was modified by Richards and Elms[9] for the design of gravity retaining walls. Solution for upper bound permanent displacement for several natural ground motions was used in this model to compute the permissible displacement under dynamic condition. Dynamic active earth force is given by,

$$P_{ae} = 0.5\gamma H^2 \frac{(1 \pm k_v)\cos^2(\phi - \theta - \alpha)}{\cos\psi \cos^2\alpha \cos(\delta + \alpha + \theta)}$$

$$\times \left[\frac{1}{1 + \left\{\frac{\sin(\phi + \delta)\sin(\phi - i - \theta)}{\cos(\alpha - i)\cos(\delta + \alpha + \theta)}\right\}^{0.5}}\right]^2. \tag{10}$$
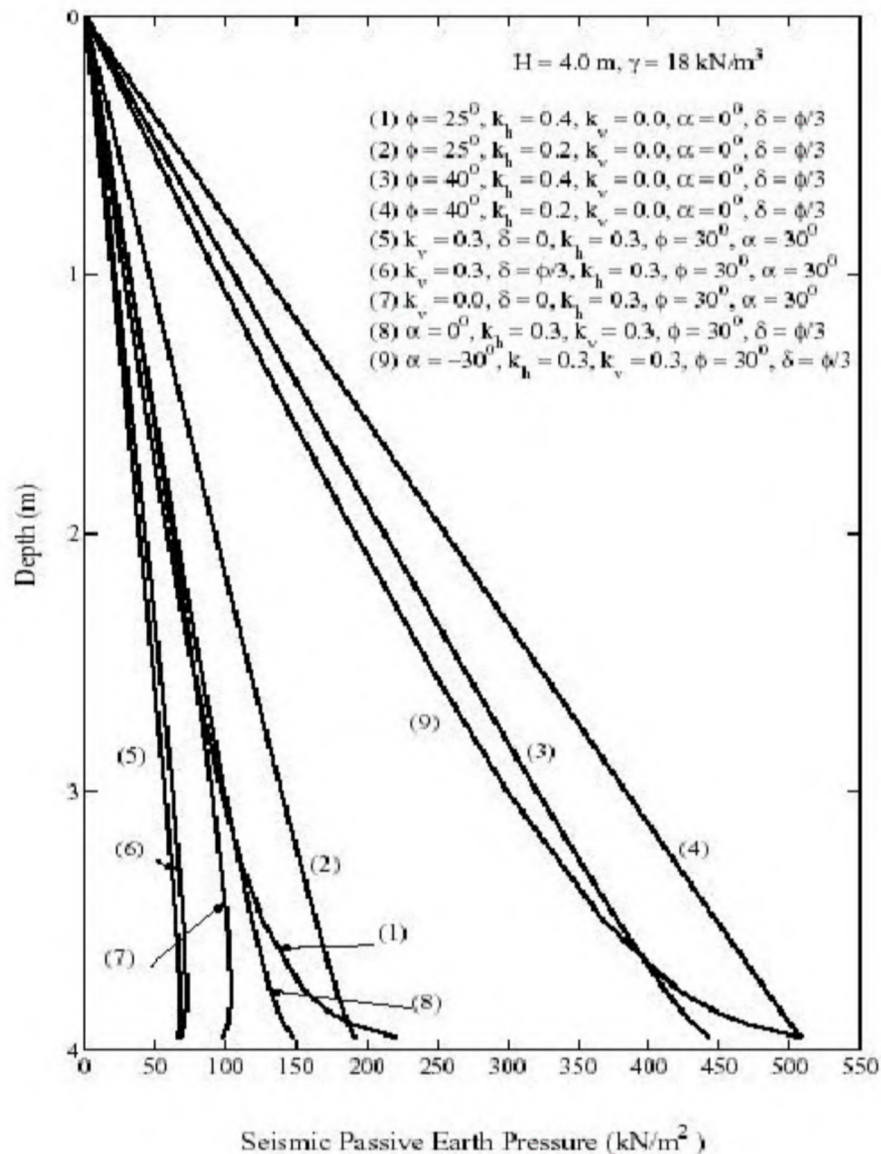
Figure 3. Typical result for point of application of seismic passive resistance[15].

The permanent block displacement is given by

$$d_{\text{perm}} = 0.087 \frac{v_{\text{max}}^2 a_{\text{max}}^3}{a_y^4}, \tag{11}$$

where v_{max} and a_{max} are the peak ground velocity and accelerations respectively. a_y = yield acceleration for the backfill-wall system and given by,

$$a_y = \left[\tan\phi_b - \frac{P_{\text{ae}}\cos(\delta + \theta) - P_{\text{ae}}\sin(\delta + \theta)}{W} \right] g. \tag{12}$$

The mathematical model proposed by Wu[12] for SDOF mass-spring-dashpot system based on soil-wall mass, damping in the system and soil stiffness gives the solution for retaining wall subjected to translational and rotation about base combined mode of movement. The mathematical model proposed by Nandakumaran[17] for SDOF mass-spring-dashpot system, based on soil-wall mass, damping in the system and soil stiffness gives the solution for retaining wall subjected to translational mode of movement. Later, Prakash[10] extended the solution for the case of pure rotation of the wall. Nadim and Whitman[11] used a two-dimensional plane-strain finite element model for computing permanent displacements taking into account the ground amplification. Reddy

et al.[18] proposed a mathematical model based on soil springs to consider translational and rotational movements of wall.

Codal provisions for design

According to Indian Standard code of practice IS 1893 (ref. 19), the stability of the retaining wall is calculated based on pseudo-static approach. The earth pressures under seismic conditions are computed using Mononobe–Okabe method of analysis. Dynamic increment of the earth pressure is arbitrarily assumed to act at mid-height of the wall. Effects of dry, partially submerged and saturated backfill are considered in the analysis. But the permissible displacement is not specified as this code used force-based analysis proposed by Mononobe–Okabe. In Eurocode 8 (ref. 20), the pseudo-static analysis is considered using Richards and Elms model. Hence the permissible displacement for translation and rocking mode are considered in the analysis. Nonlinear behaviour of base soil and backfill are considered in this code. But here also the point of application of the dynamic increment is assumed at mid-height of the wall. Hydrodynamic force for saturated backfill is assumed to act at $0.4H$ from the base of the wall with height H.

Differences in several methods

The numerical example in Table 1 shows the differences in various methods for seismic earth pressure and its point of application calculation. Mononobe–Okabe method gives maximum value of seismic earth pressures both in active and passive case. Hence though in active case, it is on the safer side, but in passive case it is on the unsafe side. Hence for passive case in forced-based analysis, Choudhury[6] gives reasonable design value of the seismic earth pressure along with the point of application based on an analytical model. But overall, the displacement-based analysis gives not only the reasonable value of seismic earth pressures but also the permissible displacement is considered.

Seismic design of earth dam

The stability analysis of earth dams is done as per IS 7894, which basically adopts a pseudo-static approach. By defini-

tion, factor of safety of a slope is the ratio of shear strength to shear stress. The computation of shear stress is dependent on appropriate consideration of seismic forces. The shear strength can be expressed either in terms of total stresses or in terms of effective stresses. Depending on the choice, the stability analysis is termed as total stress analysis or effective stress analysis. The total stress analysis is ideally suited for earthquakes, because there is a change in shear stress which occurs quickly enough that soft cohesive soil does not have time to consolidate; or in the case of heavily over-consolidated cohesive soils, the negative pore water pressures do not have time to dissipate. A major disadvantage of this approach is that the accuracy of the undrained shear strength is always in doubt because it depends on the shear-induced pore water pressures (which are not measured), which in turn depend on the quality of field samples and laboratory test procedures.

The effective stress analysis uses the drained shear strength parameters (c' and ϕ'). The analysis could be used for earthquake-induced loading, provided the earthquake-induced pore water pressures can be estimated. In other words, the effective stress generated during earthquake must be determined. Since the shear strength is directly related to the effective stress, this method of analysis is more rational. However, the pore water pressures must be included in the analysis, whose accuracy is often in doubt.

The factor of safety (F.S.) under earthquake condition can be calculated using the following equation,

$$\text{F.S.} = \frac{\sum(c'\Delta L + (N-U)\tan\phi') - \sum(W\sin\theta\tan\phi' A_\text{H})}{\sum W\sin\theta + \sum W\cos\theta A_\text{H}}, \quad (13)$$

where c' = effective cohesive resistance per unit length; N = force normal to the arc of the slice; U = force due to pore water pressure; $(N-U)$ = effective normal force acting on the failure surface of slice; ϕ' = effective angle of internal friction; W = weight of the slice considered for driving force; θ = angle between vertical through the centre of the slice and radius of failure surface; and A_H = horizontal seismic coefficient.

Correctly speaking the internal force should be taken at the centroid of the slice. Taking it at the base simplifies computations and gives somewhat conservative results.

Table 1. Computation of seismic earth pressures by various methods

Methods	Mononobe–Okabe	Choudhury	Steedman and Zeng	Richards and Elms	IS1893	Eurocode 8
Seismic active earth pressure, P_ad (kN/m)	145.16	–	141.24	100.92	145.16	100.92
p.o.a. of P_ad from base (m)	2.72	–	2.00	2.33	2.72	2.33
Seismic passive earth pressure, P_pd (kN/m)	943.99	905.36	–	–	943.99	–
p.o.a. of P_pd from base (m)	–	1.98	–	–	–	–
Displacement (mm)	–	–	–	100	–	100
Remarks	A	A	A	B	A	B

Data used: $H = 6$ m, $c = 0$, $\phi = 34°$, $\delta = 17°$, $\gamma = 17.3$ kN/m³, $k_\text{h} = 0.3$ and $k_\text{v} = 0.3$.
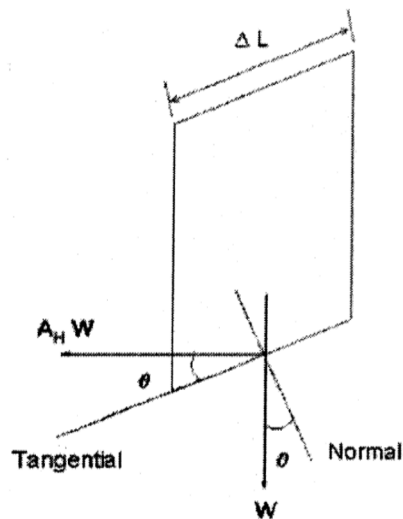A = Force-based analysis and B = Displacement-based analysis.

Figure 4. Forces acting on each slice for analysis of earth dam.

Referring to Figure 4, it can be seen that it is in terms of effective strength parameters, c' and ϕ' and pore water pressure. For the pseudo-static method, it is common to assume that the same pore water pressures exist for the static case and the pseudo-static case. Though eq. (13) is written as a conventional Fellenius equation, the analysis can be carried out by any other refined method like that of Bishop or Morgenstern and Price. The critical condition of analysis of upstream slopes of an earth dam for operating condition is the sudden draw down. When the reservoir is full the seepage pressure acting towards the downstream side increases the resistance of upstream slope towards the sliding and as such, such a condition is not considered critical. However, this condition combined with earthquake forces is considered as critical as the stability of upstream slope gets reduced and may lead to the failure of the dam when the reservoir is full. Similarly, the downstream slope of the dam shall be analysed for a condition of steady seepage combined with earthquake forces. The minimum desired values of factor of safety under these conditions are 1.0 as per IS 7894.

Seismic-bearing capacity of shallow footings

Bearing capacity of shallow foundations is an important topic of research in geotechnical engineering. Prandtl[21], Terzaghi[14], Meyerhof[22], Hansen[23], Vesic[24] and many others have extensively studied the bearing capacity of shallow footings under the static case. But a limited number of research works was done in the area of seismic-bearing capacity of footings. As a pioneering work, Meyerhof[25,26] did the analysis for seismic-bearing capacity factors by considering the application of pseudo-static seismic forces at the structure only as inclined pseudo-static loads on the foot-

ing. The effect of seismic forces on the inertia of the supporting soil was not, however, considered. Later, correcting this error, a few researchers did the analysis for seismic-bearing capacity of shallow footings as discussed below.

Sarma and Iossifelis[27] determined the seismic-bearing capacity factors for shallow strip footings by using the limit equilibrium method of slope stability analysis with inclined slices. They considered the pseudo-static effect of inertia of soil mass underneath a footing in terms of horizontal seismic acceleration.

Using a Coulomb-type mechanism, i.e. with planar rupture surfaces including the pseudo-static inertia forces in the soil and on the footing, seismic-bearing capacity factors were determined by Richards et al.[28]. They had examined field and laboratory observations of seismic settlements of shallow foundations on granular soils that were not attributable to change in density in terms of seismic degradation of bearing capacity. Planar rupture failure mechanisms were considered in the analysis. Sliding block procedure was used in the analysis with the assumption of different mobilized wall friction angles δ. This procedure was used for computing settlements due to loss of bearing capacity under seismic conditions. The approach also leads to a design procedure for footings based on limiting seismic settlements to a prescribed value.

Budhu and Al-Karni[29] derived the seismic-bearing capacity factors for general c–ϕ soil by considering both the horizontal and vertical seismic accelerations. A composite failure surface with pseudo-static forces was considered in the analysis. Mohr–Coulomb failure theory was assumed to determine the inclinations of the failure planes. The limit equilibrium solutions were described for evaluating the seismic-bearing capacity factors from static one. The focus of the logspiral portion of the composite failure surface was assumed at the edge of the footing.

Seismic-bearing capacity factors of a strip surface footing resting on a cohesionless soil were determined by Dormieux and Pecker[30]. The upper bound theorem of the yield design theory was used to obtain the ultimate load. Normal and tangential forces applied to the foundation and the inertia forces developed within the soil volume were considered. The classical Prandtl-like mechanism was used to show that the reduction in the bearing capacity was mainly caused by the load inclination.

Based on the kinematical approach of yield design theory, seismic-bearing capacity of shallow strip foundations on dry soils was analysed by Paolucci and Pecker[31]. Seismic effects on the bearing capacity of shallow foundations on soils obeying Mohr–Coulomb failure criterion were investigated both theoretically and experimentally. The curved failure surface and the kinematic mechanism were considered in the analysis. The effect of load inclination, eccentricity of seismic actions and soil inertia were considered in the analysis. It had been shown that the effect of load inclination is much more than the seismic effect on soil inertia to reduce the bearing capacity factors. Load eccentricity

also plays an important role in the pseudo-static approach to give very high value of safety factors in the design of shallow footings.

Seismic-bearing capacity factors of shallow strip footings were calculated by considering pseudo-static seismic forces. Soubra[32] used upper bound limit analysis by considering two non-symmetrical failure mechanisms called M1 and M2. M1 consisted of a log sandwich composed of a triangular active wedge, a logspiral radial shear zone and a triangular passive wedge. M2 consisted of an arc sandwich composed of a triangular active wedge, a circular radial shear zone and a triangular passive wedge. The results were obtained using rigorous upper-bound limit analysis theory with an associated flow rule for a Coulomb material. The seismic-bearing capacity factors were presented in the form of design charts for practical use.

Soubra[33] did the analysis for shallow strip footings with two kinematically admissible failure mechanisms M1 and M2. Here M1 was symmetrical and composed of a triangular active wedge under the footing and two radial shear zones composed of a sequence of rigid triangles. M2 was non-symmetrical and composed of a single radial shear zone. Pseudo-static representation of earthquake forces with the use of upper bound limit analysis theory was adopted.

Kumar and Rao[34] did the pseudo-static analysis for spread footing by employing the method of stress characteristics. Seismic-bearing capacity factors were plotted for various horizontal earthquake acceleration coefficients and soil friction angles. Both the single-sided and double-sided failure mechanisms were considered in the analysis. The closed form solution for cohesion and surcharge components, N_c and N_q respectively, were found and compared with those obtained by other researchers. The N_γ values were found to be significantly smaller than those obtained from other theories. The effects of vertical seismic accelerations were not considered in the analysis. The nature of the pressure distribution below the footing and the failure patterns were also shown with the variation of seismic earthquake accelerations.

Very recently, Choudhury and Subba Rao[35] have established the minimum seismic-bearing capacity factors N_{cd}, N_{qd} and $N_{\gamma d}$ with respect to cohesion, surcharge and unit weight components for a shallow strip footing using limit equilibrium method of solution. One-sided failure mechanism with partial mobilization of passive pressure on rarer side of the failure was considered in the pseudo-static analysis. A search for the critical focus of the logspiral was considered to develop minimum design values of the seismic-bearing capacity factors. Figure 5 shows the typical comparison of all the available methods to compute the seismic-bearing capacity factors. The seismic-bearing capacity (q_{ud}) can be calculated using the following equation,

$$q_{ud} = cN_{cd} + qN_{qd} + 0.5\gamma BN_{\gamma d}, \tag{14}$$

where c = unit cohesion, q = surcharge at foundation base level, γ = unit weight of the soil, B = width of the foundation.
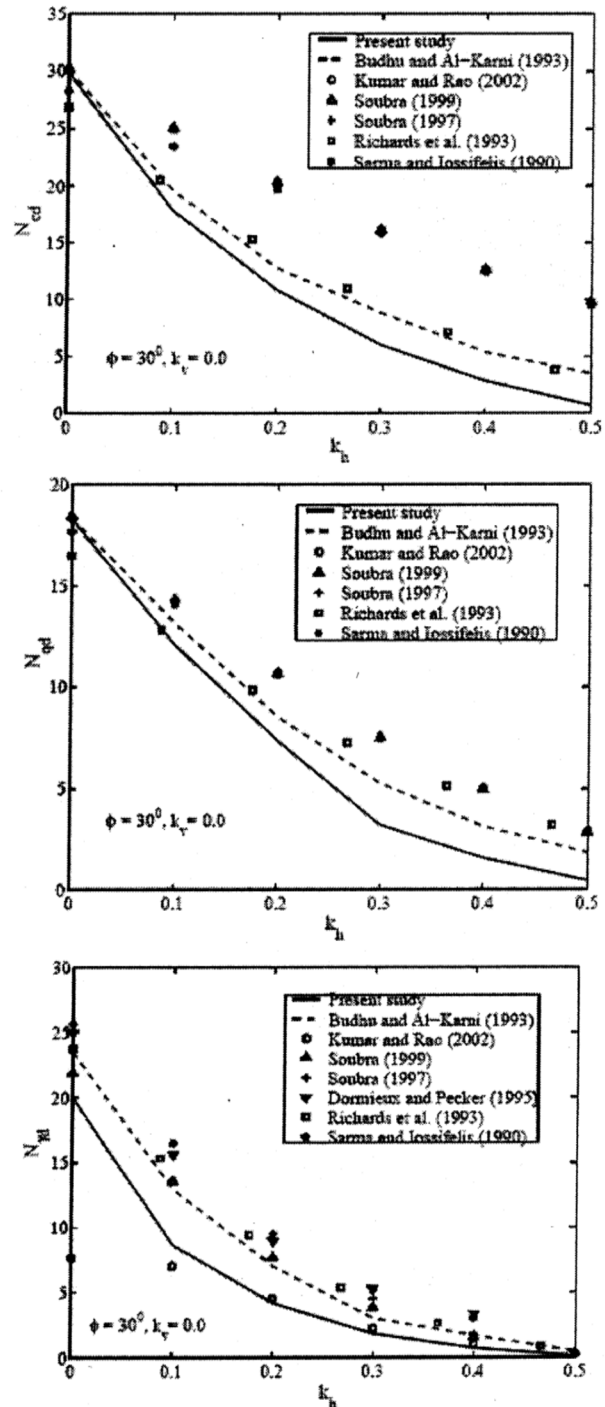

Figure 5. Typical comparison of different methods to compute seismic-bearing capacity factors (ref. 35).

Earthquake mitigation techniques

Retaining wall

For design of retaining wall under earthquake condition, the method which gives the maximum active earth pressure and minimum passive earth pressure should be considered. Also the point of application of the seismic earth pressure must be considered based on some logical analysis, rather than some arbitrary selection of value. In displacement-based analysis, wall dimensions are determined for given factors of safety under static condition (Sliding = 1.5, Overturning = 1.5, Bearing capacity = 2.5, Eccentricity = 1/6th of the base size). Cumulative displacements and rotations of wall then must be computed for different loading cycles (magnitude of earthquake) for a given ground motion. The computed displacements are compared with the permissible displacements. If the computed displacement is more than the permissible displacement, wall section will be redesigned for the permissible displacement.

Earth dam

The design under earthquake condition can be done using the method described above. The measures that can be taken to mitigate the earthquake hazards for earth dam designs are: ground improvement, use of more impervious soil, factored strength parameters and earthquake-induced pore water pressures.

Foundations

Among the available methods to compute the seismic-bearing capacity of shallow foundations, for the design purpose, the method which gives the minimum value of the bearing capacity of the soil under a particular earthquake condition should be considered. As per the reduced bearing capacity from the static condition, the section required for the foundation has to be determined. If the anticipated settlement of foundation under earthquake conditions exceeds the permissible limit, foundation alternatives like combined footing or raft footing instead of isolated footing are preferred which can reduce the bearing pressure. Site improvement techniques like dynamic compaction, dewatering, vibroflotation, etc. and use of geosynthetics or geocell embedded foundations[36,37] are the possible earthquake mitigation techniques for design of shallow foundations.

Summary

The present study shows that for the earthquake-resistant design of retaining wall, displacement-based analysis should be used for a better design rather than using the force-based analysis. For passive case of earth pressure, analysis by considering curved rupture surface should be used. Point of application of seismic earth pressure should be computed based on some logical analysis instead of some arbitrary selection. In this context, it is sure that for safe and economic design of retaining walls under both active and passive conditions, Indian Standard code (IS 1893) should be modified properly. In seismic design of earth dam, currently pseudo-static methods are used, which excludes the deformation criteria. Here, factored shear strength and pore pressure values must be used in the design of earth dam. Earthquake-resistant design of foundations can be carried out by estimating minimum design value of bearing capacity using the available methods in the literature. To mitigate earthquake hazards, different techniques can be adopted as alternative foundation.

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