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Estimation of freeboard requirements against overtopping of surface impoundments under seismic action

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Abstract The excitation of structural components and liquid contents of surface impoundments by seismic waves can generate turbulence that is large enough to overtop the bounding berms. In cases in which the liquids are wastes from industrial/municipal operations, their release from impoundments can pose significant risks to the environment. In this analysis, the freeboard magnitudes that can accommodate liquid head levels in impoundments are determined through linkage of configuration of waves in the liquid surface to incident seismic wave characteristics, liquid characteristics and impoundment design. For an impoundment site in a region of ground acceleration levels ranging from 0.2 to 1.0 g and impacted by seismic shear wave velocity of 180 m/s, freeboard requirements are in the range of 0.004–2.0 m on soft soil; 0.008–0.7 m on medium-dense soil; and 0.002–0.1 m for dense soil. For the same impoundment design, ground acceleration and incident wave characteristics, freeboard requirements are directly proportional to the depth of the soil mantle over bedrock. The impoundment slope, which is a key parameter with regards to liquid holding volumetric capacity of the impoundment, is a less significant parameter than depth to bedrock with regard to the size of the required freeboard. This implies that siting of an impoundment should be considered to be critical to impoundment performance in seismic zones.

Keywords Impoundment · Waste containment · Seismic · Freeboard · Overtopping

Introduction

The United States Environmental Protection Agency (US EPA) defines a surface impoundment as a natural topographic depression, artificial excavation, or dike arrangement for storing, treating, or disposing of wastewaters (liquids or semi-solid waste with less than 5% solids by weight). They are used in wastewater and stormwater detention, treatment, and disposal in agriculture, mining, chemical processing and other industrial operations. Typically, surface impoundments are multicomponent and multi-layered waste containment structures (Fig. 1 is a schematic illustration of typical surface impoundment showing configuration parameters). The structure may be constructed above the ground, below the ground, or may be partly embedded in the ground. Generally, its configuration is such that the length or width is greater than its depth.

According to the US EPA (2002), there are more than 30,000 surface impoundments that receive wastewater and stormwater runoff in the United States. In the United States alone, industrial surface impoundments are used to manage more than 650 million metric tons of industrial wastewater; more than 20 million people live within 2 km of industrial surface impoundments; more than 20% of surface impoundments are within 150 m of

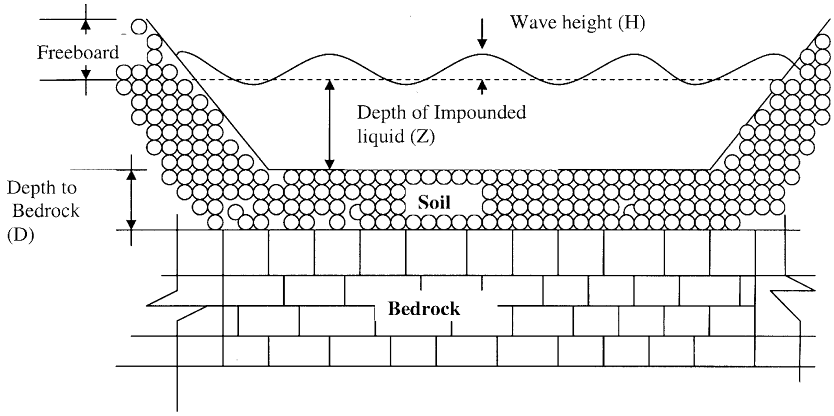


Fig. 1 Surface impoundment showing configuration parameters

drinking water systems, fishable water bodies and a few meters above groundwater; and more than half of industrial surface impoundments contain at least one or more of the 256 chemicals of concern with respect to human health and/or have either high (11–12.5) or low (2–3) pH (US EPA 2002).

Wastewater management in surface impoundments serves the following functions: treatment of wastewaters before discharge into receiving surface water; storage of excess wastewater; and evaporation and/or seepage of wastewater into the ground. Among the structural failures that impact upon the functional performance of surface impoundments are berm failures and overtopping of contained wastewater due to seismic action. US EPA (2002) reports that 25% of surface impoundments in the US have been reported to have experienced overtopping resulting from: (1) extreme seismic event; (2) flood event; and (3) changes in wastewater characteristics due to changes in processing practices. The US EPA Science Advisory Board identified these failure conditions in its report (US EPA 2002). However, due to the complexity and uncertainties involved in analyzing these conditions and their linkage to risk assessment models, potential human health and ecological exposure that may result from these abnormal conditions have not yet been addressed quantitatively in most risk frameworks for estimating the reliability of hazardous surface impoundments.

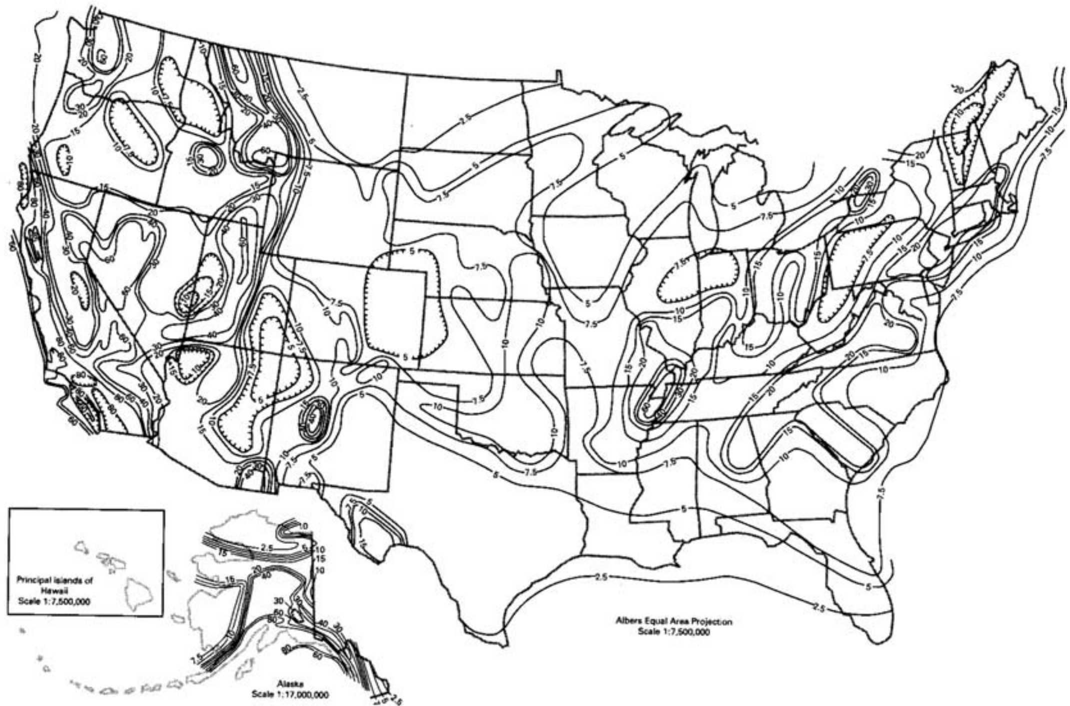
Source term concentrations are perhaps the most sensitive parameters in ground water and surface water contamination risk assessment. Quantification of source term concentrations of contaminants under abnormal operating conditions of surface impoundments is challenging and plausibly the reason for the lack of comprehensive methodologies for risk assessment of surface impoundments under abnormal operating conditions such as transient events. As the characteristics of transient events are region-specific (for example earthquakes in west and central USA and storm-induced flood in the southeast, USA), structural damages to various components of surface impoundments as a result of transient events exemplified by earthquakes will vary in magnitude. Then, it is logical that any methodology for quantifying overtopping or specifying freeboard requirements against overtopping in surface impoundments under transient events be based on regionalized data.

Presently, the practice in the design of surface impoundments is to provide a minimum of 0.6 m freeboard to prevent overtopping. This practice may be adequate for surface impoundments in regions characterized by low ground shaking and low frequency of high magnitude floods; however, it is inadequate in regions that are vulnerable to high-level ground shaking. On the other hand, providing excessive freeboard will be uneconomical especially for surface impoundments that cover several hundreds of hectares of land. Table 1 shows the statistics of impoundment surface areas in the US. Figure 2a, b are maps that show magnitudes of seismic events in terms of ground accelerations, and the distribution of surface impoundments in various regions of the United States, respectively. An inspection of both figures indicates that there is significant overlap of geographical distribution of impoundments and regions of high probability of significant ground shaking.

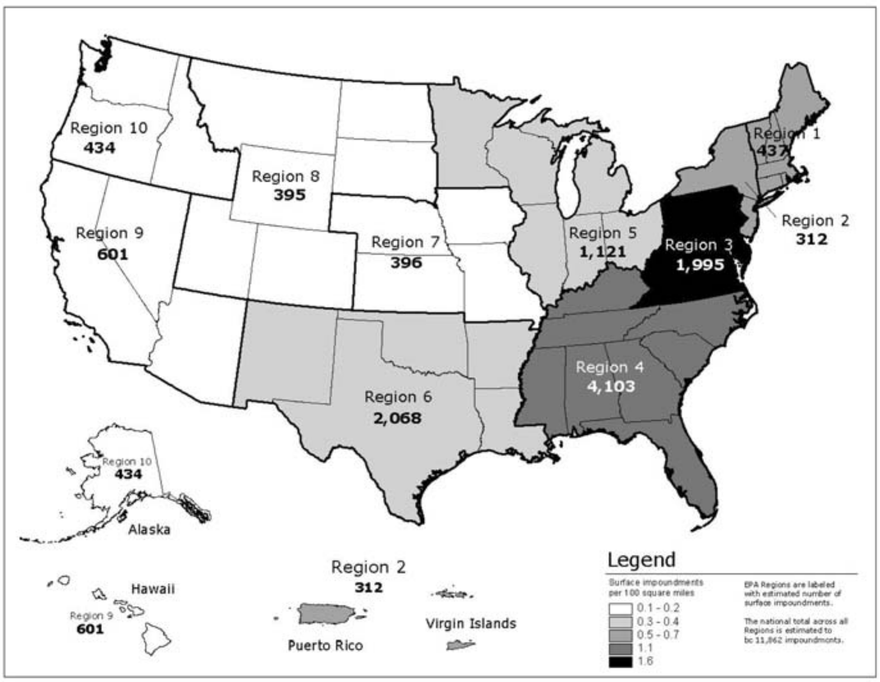
There is a dearth of information on the mechanics of seismically-induced liquid overtopping of impoundments. Johnson and Anderson (1987) used the principle of wave run-up on coastal structures to quantify

Table 1 Statistical distribution of impoundment surface area in the US (US EPA 2002)

Size range (hectares)	Impoundment surface area (%)
0–0.25	51
0.25–1	25
1–5	17
5–10	4
10–500	4



(a)



(b)

Fig. 2 **a** Seismic impact zones in the US (adapted from the US EPA 1993), and **b** distribution of hazardous waste impoundments in the US (adapted from US EPA 2001) the freeboard requirements for hazardous waste impoundments against wind-induced overtopping. An adequate methodology for computing freeboard requirements under seismic action should involve integration of the mechanics of transient action of seismic shaking of impoundments with those of wave generation and transport under the constraints of the berm of the impoundments.

Advances in coastal engineering have led to the development of several empirical equations that are fitted to data obtained from laboratory experiments that are performed to estimate the magnitude of wave run-up on beaches. Some of the scenarios covered are adaptable to the analysis of surface impoundments. In this paper, these expressions are modified as cited, to cover surface impoundments by introducing seismic parameters as wave generators, and adapting dimensional parameters to the typical design configurations of surface impoundments. These analyses are generalized to impoundments of various design configurations so that the design equations can be applied to any impoundments if site characteristics, impoundment component design and content characteristics are known. Scaled charts are provided for typical ranges of parameter magnitudes. A design example is also provided. This work is part of a larger study on quantification of potential damages to waste containment systems due to seismic events in the US intended for introduction into the multi-hazard loss estimation methodology (HAZUS-MH). The methodology was developed under the auspices of the Federal Emergency Management Agency (FEMA).

Overview of techniques for determining wave run-up and overtopping of coastal structures

Freeboard requirements for coastal structures against overtopping are mostly determined on the basis of wave run-up, which is the vertical distance to which the liquid contained will rise above a stationary liquid level on the slope of the embankment. The magnitude of wave run-up on beaches and coastal structures depends on the characteristics of the wave (e.g., wave height, period, energy, etc.). Wave run-up on smooth embankment slopes is typically higher than those on rough slopes because energy is dissipated rapidly in the latter. Equation 1 developed by Wassing (1957) is the oldest method of estimating run-up generated by irregular waves on smooth impermeable slopes.

$$\mathrm{Ru}_{2\%} = 8H_s \tan\alpha. \tag{1}$$

In Eq. 1, $\mathrm{Ru}_{2\%}$ is defined as the run-up level exceeded by 2% of the expected run-up height (L); α is the slope of the structure (°); and H_s is the wave height (L). The parameter, H_s in equations for computing the wave run-up is often taken to be the deepwater wave height. Expressions that are similar to Eq. 1 were later developed by researchers including Hunt (1959) who formulated Eq. 2 for regular wave run-up.

$$\frac{R}{H} = \frac{\tan\alpha}{\sqrt{2\pi H/gT^2}}. \tag{2}$$

The parameter T in Eq. 2 is the period of the wave (T) and R is the wave run-up (L). Battjes (1974) extended Eq. 2 to cover run-up generated by irregular wave as in Eq. 3.

$$\frac{R}{H} = \frac{C\tan\alpha}{\sqrt{2\pi H/gT^2}}. \tag{3}$$

The coefficient, C, in Eq. 3 is dependent on the stage of development of the sea and its magnitude is estimated to be 1.49 for fully developed seas and 1.87 for young seas. Chue (1980) developed Eq. 4 for computing wave run-up for breaking and non-breaking waves by fitting the equation to a wide range of wave and slope conditions.

$$\frac{R}{H_o} = 1.8\left(1.0 - 3.111\frac{H_o}{L_o}\right)\left(\frac{\pi}{2\theta}\right)^{1/2}, \tag{4}$$

where

$$L_o = \frac{gT^2}{2\pi}. \tag{5}$$

In Eq. 4, R is defined as the maximum elevation of wave uprush above the still water level on a smooth slope (L); H_o is the unrefracted deep water wave height (L); θ is the angle between the run-up slope and the horizontal (radians); and L_o is the deep water wave length (L). Equation 4 is based on the assumption that the relative run-up, R/H_o decreases as the wave steepness, H_o/L_o increases. However, later analyses by Ahrens and Titus (1985) based on data from Savage (1958), show that the relative run-up increases with increase in wave height, period, and nonlinearity; and therefore proposed Eqs. 6, 7, and 8 for non-breaking waves, plunging waves, and waves within the transition region.

$$\frac{R}{H_o} = c_o\left(\frac{\pi}{2\theta}\right)^{c_1} \exp\left[c_2\left(\frac{\eta_c}{H} - 0.5\right)^2\right]. \tag{6}$$

$$\frac{R}{H_o} = \frac{0.967\tan\alpha}{\sqrt{2\pi H/gT^2}}. \tag{7}$$

$$\frac{R}{H_o} = c_o\left(\frac{\xi - 2.0}{1.5}\right)\left(\frac{\pi}{2\theta}\right)^{c_1} \exp\left[c_2\left(\frac{\eta_c}{H} - 0.5\right)\right]^{c_2} + 0.967\left(\frac{3.5 - \xi}{1.5}\right)\frac{\tan\alpha}{(2\pi H/gT^2)^{1/2}}. \tag{8}$$

$$\xi = \frac{\tan\alpha}{\sqrt{2\pi H/gT^2}}. \tag{9}$$

In Eqs. 6 through 8, η_c is defined as the crest of the wave above the still water level (L); H is the wave height at the toe of the slope (L); ξ is the surf similarity parameter defined quantitatively as in Eq. 9; and c_1 and c_2 are dimensionless coefficients (–). Equation 7 is similar to Eq. 3 by Battjes (1974) with the coefficient, C in the latter being equal to 0.967. The condition of the wave (standing, plunging, or transition region) is defined by the magnitude of the surf parameter as follows: stationary condition, $\xi \geq 3.5$; plunging condition, $\xi \leq 2.0$; and transition region, $2.0 < \xi < 3.5$. Apparently, Eq. 8 for the transition region is based on the weighted average of Eq. 6 and 7. At present, research is still on-going on the identification of the most critical factors affecting wave run-up and overtopping of coastal structures. However, Eq. 2, which is the fundamental expression upon which latter equations are based, shows that the magnitude of the wave run-up is proportional to the wave period, slope, and square root of the wave height.

Hughes (2004) formulated a set of equations for estimating wave run-up for breaking and non-breaking waves in terms of a dimensionless parameter referred to as the maximum depth-integrated momentum flux, M_F. The magnitude of M_F varies over the wavelength from large positive values at the crest to large negative values at the trough. The parameter, M_F is formulated by assuming that the equation for linear wave kinematics is valid in the crest region of the wave. Equations 10 and 11 were subsequently developed for estimating the magnitude of M_F for cases of regular sinusoidal and peaked crest shallow waves, respectively.

$$\left(\frac{M_F}{\rho g h^2}\right)_{max} = \frac{1}{2}\left(\frac{H}{h}\right)\frac{\sinh[k(h+H/2)]}{kh\cosh(kh)} + \frac{1}{8}\left(\frac{H}{h}\right)^2\left[\frac{\sinh[2k(h+H/2)+2k(h+H/2)]}{\sinh 2kh}\right], \tag{10}$$

$$\left(\frac{M_F}{\rho g h^2}\right)_{max} = A_o\left(\frac{h}{gT^2}\right)^{-A_1}, \tag{11}$$

where

$$A_o = 0.6392\left(\frac{H}{h}\right)^{2.0256}. \tag{12}$$

$$A_1 = 0.1804\left(\frac{H}{h}\right)^{-0.391}. \tag{13}$$

In Eqs. 10, 11, 12, and 13, h is the water depth (L). Equations 11, 12, and 13 were obtained empirically. A similar expression was developed for computing M_F for the case of solitary waves (Eq. 14).

$$\left(\frac{M_F}{\rho g h^2}\right)_{max} = \frac{1}{2}\left[\left(\frac{H}{h}\right)^2 + 2\left(\frac{H}{h}\right)\right] + \frac{N^2}{2M}\left(\frac{H}{h}+1\right)\left\{\tan\left[\frac{M}{2}\left(\frac{H}{h}+1\right)\right] + \frac{1}{3}\tan^3\left[\frac{M}{2}\left(\frac{H}{h}+1\right)\right]\right\}, \tag{14}$$

where

$$M = 0.98\left\{\tanh\left[2.24\left(\frac{H}{h}\right)\right]\right\}^{0.44}. \tag{15}$$

$$N = 0.69\tanh\left[2.38\left(\frac{H}{h}\right)\right]. \tag{16}$$

Previous work by Archetti and Brocchini (2002) shows that the magnitude of wave run-up on an impermeable slope is directly proportional to the maximum depth-integrated wave momentum flux, prior to impacting on the toe of the slope. Based on this theory, Hughes (2004) formulated Eq. 17 by equating the weight of liquid above the still level to the maximum depth-integrated wave momentum flux, M_F.

$$\frac{R}{h} = \left(\frac{2K_p\tan\alpha}{K_M\left[\frac{\tan\alpha}{\tan\theta}-1\right]}\right)^{1/2}\left(\frac{M_F}{\rho g h^2}\right)^{1/2}. \tag{17}$$

In Eq. 17, K_P is a reduction factor for slope porosity, its magnitude being unity for an impermeable slope; and K_M is a constant of proportionality. Eq. 17 was reduced to simpler equations (Eqs. 18, 19a, and 19b) by Hughes (2004) for the case of regular wave, breaking and non-breaking solitary wave run-up, respectively, by fitting them to series of laboratory data for each scenario.

$$\frac{R}{h} = 3.84\tan\alpha\left(\frac{M_F}{\rho g h^2}\right)^{1/2}. \tag{18}$$

$$\frac{R}{h} = (1.39 - 0.027\cot\alpha)\left(\frac{M_F}{\rho g h^2}\right)^{1/2}. \tag{19a}$$

$$\frac{R}{h} = 1.82(\cot\alpha)^{1/5}\left(\frac{M_F}{\rho g h^2}\right). \tag{19b}$$

Expressions were also developed for the case of irregular wave run-up with different impermeable slope angles (Hughes 2004). Owen (1982) performed a series of tests to determine overtopping discharges for various coastal structures under different magnitudes of random waves, and summarized the results in the form of Eq. 20. Equation 20 is in the form that can be used to compute the minimum freeboard required to limit liquid overtopping of impoundments to acceptable levels.

$$\frac{Q}{T_{\mathrm{m}} g H_{\mathrm{s}}} = A \exp\left(-B \frac{R_{\mathrm{c}}}{T_{\mathrm{m}}\sqrt{gH_{\mathrm{s}}}}\right). \tag{20}$$

In Eq. 20, Q is the overtopping discharge per unit length of the structure ($L^3\ T^{-1}\ L^{-1}$); H_s is the significant height of the incident waves (L); T_m is the mean zero crossing wave period (T); R_c is the freeboard (L); and A and B are constants, the magnitudes of which are dependent on slope angle, and are provided by Owen (1982). Rearranging Eq. 20 as in Eq. 21, gives an expression for the freeboard in terms of overtopping discharge.

$$R_{\mathrm{c}} = -\frac{T_{\mathrm{m}}\sqrt{gH_{\mathrm{s}}}}{B} \ln\left[\frac{Q}{T_{\mathrm{m}} g H_{\mathrm{s}} A}\right]. \tag{21}$$

The energy of water waves is absorbed by the sea bed as well as wave breaking at the water surface, but more by the former (Longuet-Higgins 2005). A number of mathematical expressions have been developed for the quantification of the rate of energy dissipation under such conditions. In all cases, it is accepted that energy dissipation rate is directly proportional to the shear stress and velocity at the boundary. In the case of shallow water with uniform bottom slope and boundary layer thickness, Linguet-Higgins (2005) used the momentum and energy flux equations to develop a simplified analytical expression (Eq. 22) that relates the change in mean water wave surface level to the wave amplitude and water depth.

$$\zeta = -\frac{a^2}{4h}. \tag{22}$$

In Eq. 22, ζ is defined as the change in mean water surface (L); a is the amplitude of the water wave (L); and h is the depth of water (L). Equation 22 is applicable to scenarios in which the density and viscosity of the liquid in the bottom layer are the same as those of water.

Freeboard requirements of surface impoundment against wind-induced overtopping

The mathematical expression developed by Johnson and Anderson (1987) for determining the freeboard requirements for hazardous waste surface impoundments against wind-induced overtopping is based on the principle of wave run-up on coastal structures. Some of the parameters that are considered significant are fetch, liquid depth, wind speed, wave height and period, wave run-up, embankment slope roughness, and wind set-up. Fetch is defined by Johnson and Anderson (1987) as the maximum unobstructed distance across a free liquid surface over which wind can act. Equations 23, 24, and 25 were developed for computing the wave height, H; wave period, T; and wind set-up, S, respectively.

$$\frac{gH}{U_{\mathrm{a}}^2} = 0.283 \tanh\left[0.53\left(\frac{gd_{\mathrm{m}}}{U_{\mathrm{a}}^2}\right)^{0.75}\right] \times \tanh\left[\frac{0.00565\left(gF/U_{\mathrm{a}}^2\right)^{0.5}}{\tanh\left\{0.53\left(gd_{\mathrm{m}}/U_{\mathrm{a}}^2\right)^{0.75}\right\}}\right], \tag{23}$$

$$\frac{gT}{U_{\mathrm{a}}} = 7.54 \tanh\left[0.833\left(\frac{gd_{\mathrm{m}}}{U_{\mathrm{a}}^2}\right)^{0.375}\right] \times \tanh\left[\frac{0.0379\left(gF/U_{\mathrm{a}}^2\right)^{0.333}}{\tanh\left\{0.833\left(gd_{\mathrm{m}}/U_{\mathrm{a}}^2\right)^{0.375}\right\}}\right], \tag{24}$$

$$\frac{S}{F} = \frac{AU_{\mathrm{a}}^2}{gd_{\mathrm{m}}} + \left(\frac{B(U_{\mathrm{a}} - U_{\mathrm{o}})^2}{gd_{\mathrm{m}}}\right)\left[\frac{d_{\mathrm{m}}}{F}\right]^{0.5}, \tag{25}$$

where

$$U_{\mathrm{o}} = 21 \times \left(\frac{g\rho_1 \upsilon_1}{\rho_{\mathrm{a}}}\right)^{0.333}. \tag{26}$$

In Eq. 23 through Eq. 26, U_a is the wind stressor factor ($L\ T^{-1}$); F is the fetch (L); d_m is the maximum depth of the impounded liquid (L); g is the acceleration due to gravity ($L\ T^{-1}$); U_o is the wind speed ($L\ T^{-1}$); ρ_1 is the density of the liquid impounded (ML^{-3}); ρ_a is the density of air (ML^{-3}); υ_1 is the kinematic viscosity of the impounded liquid ($L^2\ T^{-1}$); and A, B are constants with magnitudes of 3.30×10^{-6} and 2.08×10^{-4}, respectively. The wave run-up, R can then be computed empirically using charts developed by Saville (1956), which relate the parameters R/H_o and H_o/gT^2 for various magnitudes of slope angles. These charts are based on overtopping of coastal structures by deep water waves, hence, Johnson and Anderson (1987) describe a procedure for use in relating the depth of the liquid impounded, d_m and the height of the wave, H, in the surface impoundment, to deep water wave parameters. Eq. 27 was subsequently proposed for computing freeboard requirements.

$$f = 1.25(R + S). \tag{27}$$

Adaptation of wave run-up estimation techniques in coastal engineering to surface impoundments

Although, seismic-induced overtopping of hazardous waste surface impoundments has been observed in several regions of the US, especially in western and central regions, impoundments have not been instrumented to monitor overtoppings. Also, controlled laboratory-based tests have not been performed to simulate overtopping of surface impoundments under seismic conditions.

Computation of the displacement of an impoundment under seismic excitation is a requirement for the estimation of the characteristics of waves generated in the liquid stored in the impoundment. Eqs. 28 and 29 developed by Zeevaert (1979) for displacements of linear flexible structures such as pipelines provide an opportunity for adaptation to impoundments.

$$(\delta_z)_x = \delta_{nz} \sin\left(\frac{2\pi}{L_n}x\right), \tag{28}$$

$$\delta_{nz} = a\left(\frac{T_n}{2\pi}\right)^2 \cos\frac{\pi}{2D}Z, \tag{29}$$

where

$$T = \frac{4D}{V_s}. \tag{30}$$

In Eqs. 28, 29, and 30, δ_{nz} is the displacement of the structure in the vertical direction (L); a is seismic induced ground acceleration (LT^{-2}); T is the vibration period of the soft soil underneath the structure; D is the total thickness of the soil to bedrock (L); Z is the depth from the ground surface to the structure, which in the case of surface impoundments is the depth of the impounded liquid (L); x is the distance along the structure from the point of reference (L); and V_s is the shear wave velocity ($L\ T^{-1}$).

In the case of a surface impoundment incorporating polymeric materials in the liner and constructed below the ground level, if it is assumed that the impounded liquid at the bottom conforms in configuration, to the sinusoidal deformation of the liner component, combining Eq. 22 and 29, and introducing $a = \delta_{nz}$ and $h = Z$ yields Eq. 31, which quantifies the magnitude of water wave height at the surface.

$$H_s = \delta_{nz} - \frac{\delta_{nz}^2}{4Z} = \frac{4Z \times \delta_{nz} - \delta_{nz}^2}{4Z}. \tag{31}$$

Substituting the magnitude of δ_{nz} (Eq. 29) into Eq. 31 yields Eq. 32.

$$H_s = \frac{4Z\left[a\left(\frac{T}{2\pi}\right)^2 \cos\left(\frac{\pi}{2D}Z\right)\right] - \left[a\left(\frac{T}{2\pi}\right)^2 \cos\left(\frac{\pi}{2D}Z\right)\right]^2}{4Z}. \tag{32}$$

Expressions for estimating the freeboard requirements of surface impoundments under seismic action can be obtained by introducing Eqs. 32 and 30 (representing wave height at the water surface and period) into expression developed for water waves. Substitution of Eqs. 32 and 30 into Eq. 2 and then simplifying yields Eq. 33:

$$F = \frac{8D^2 \tan\alpha}{\pi^2 V_s^3}\left[\frac{g\left[aV_s^2\pi^2 Z\cos\left(\frac{\pi}{2D}Z\right) - D^2a^2\cos^2\left(\frac{\pi}{2D}Z\right)\right]}{2\pi Z}\right]^{1/2}. \tag{33}$$

In Eq. 33, F is the freeboard requirement of the surface impoundments under seismic action (L). All the other parameters have been defined previously.

Generalized analysis of freeboard requirements of different configurations of surface impoundments using typical seismic and geological data for different regions of the US

As evident in Eq. 33, the required magnitude of the freeboard of a surface impoundment depends on the configuration of the impoundment (slope and depth of impounded liquid), geological parameters (depth from ground surface to bedrock and soil type), and seismic parameters (ground acceleration). The magnitude of these parameters catalogued for different regions of the US by the US Geological Survey (USGS) are used along with the expression developed for freeboard requirement developed herein to develop Figures 3, 4, 5, 6, 7, 8, 9, 10, 11. Table 2 shows ranges of magnitudes of geological, seismic, and impoundments structural configuration parameters used in developing the figures. These ranges of parameters magnitudes cover typical earthquake characteristics, soil types and surface impoundment configurations in the US Similar ranges are also used in the development of the HAZUS-MH by FEMA.

Figures 3, 4, 5 show freeboard requirements for different configurations of surface impoundments in soft soil (shear wave velocity of 180 m/s) as a function of amplified ground acceleration (0.2–1.0 g) for different magnitude of depths from ground surface to bedrock. Figures 6, 7, 8 and Figs. 9, 10, 11 show similar relationships for impoundments in medium dense soil (shear wave velocity of 360 m/s) and dense soils (shear wave velocity of 760 m/s), respectively. The magnitude of z used in generalized study is 5 m. The relationships illustrated in Figs. 3, 4, 5, 6, 7, 8, 9, 10, 11 can be incorporated into hazard loss estimation methodologies such as HAZUS-MH. It can as well be used by regulatory agencies (e.g., USEPA) for risk assessment analysis.

Table 2 Typical ranges of seismic, geological, and impoundments structural configuration parameters magnitudes in the US

Parameters	Magnitude range
Geological	
Depth to bedrock (m)	10–75
Shear wave velocity of soil (m/s)	180–760
Seismic	
Amplified horizontal ground acceleration (m/s^2)	0.1–1.0 g
Structural configuration	
Slope (°)	15–30
Depth of impounded liquid (m)	5

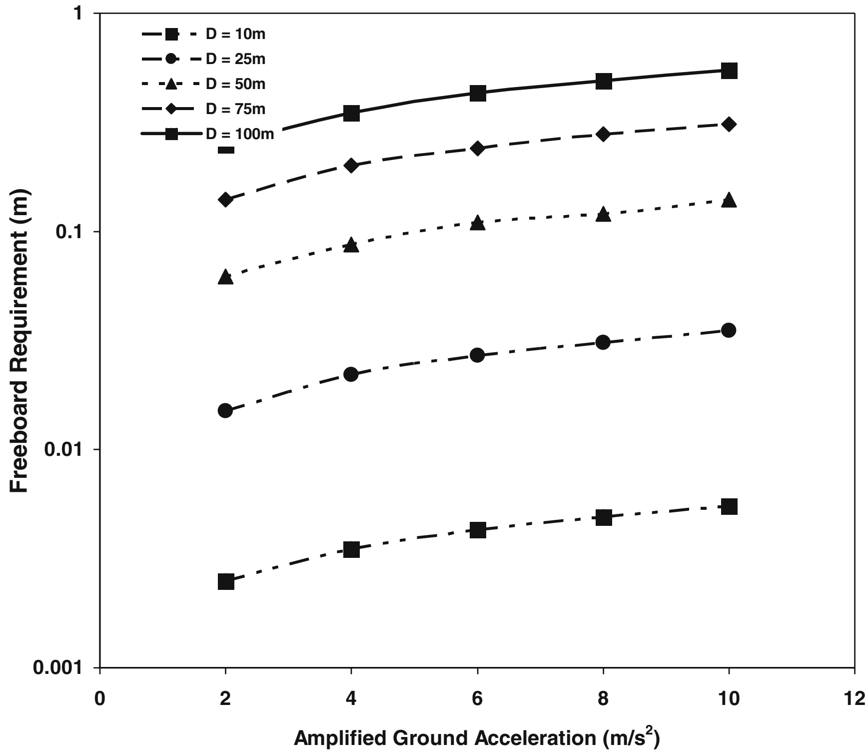


Fig. 3 Freeboard requirements of surface impoundment in soft soil under seismic action (impoundment slope is 10°)

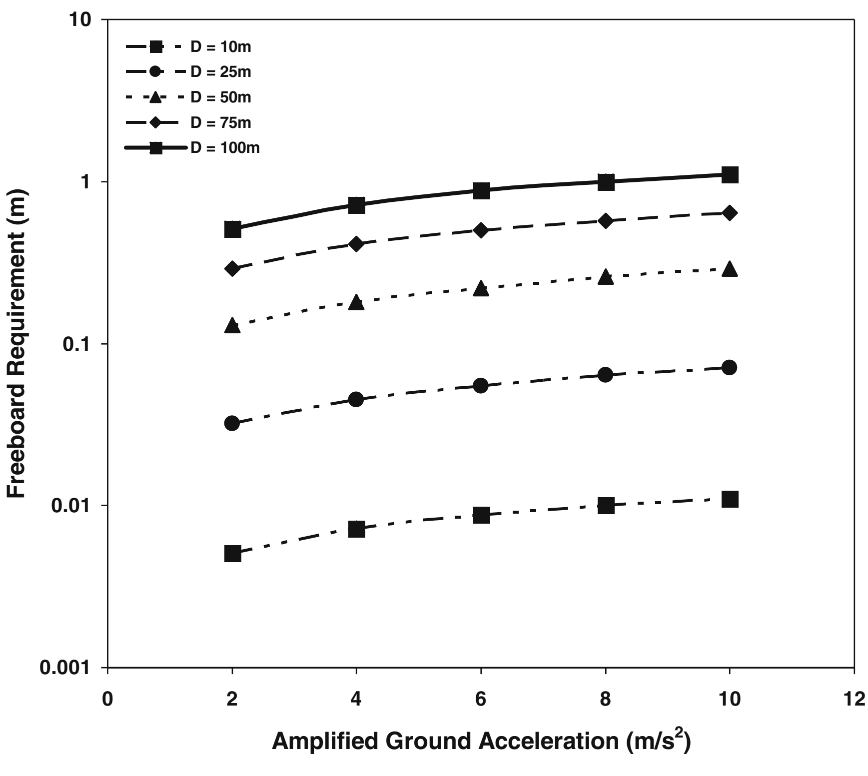


Fig. 4 Freeboard requirements of surface impoundment in soft soil under seismic action (impoundment slope is 20°)

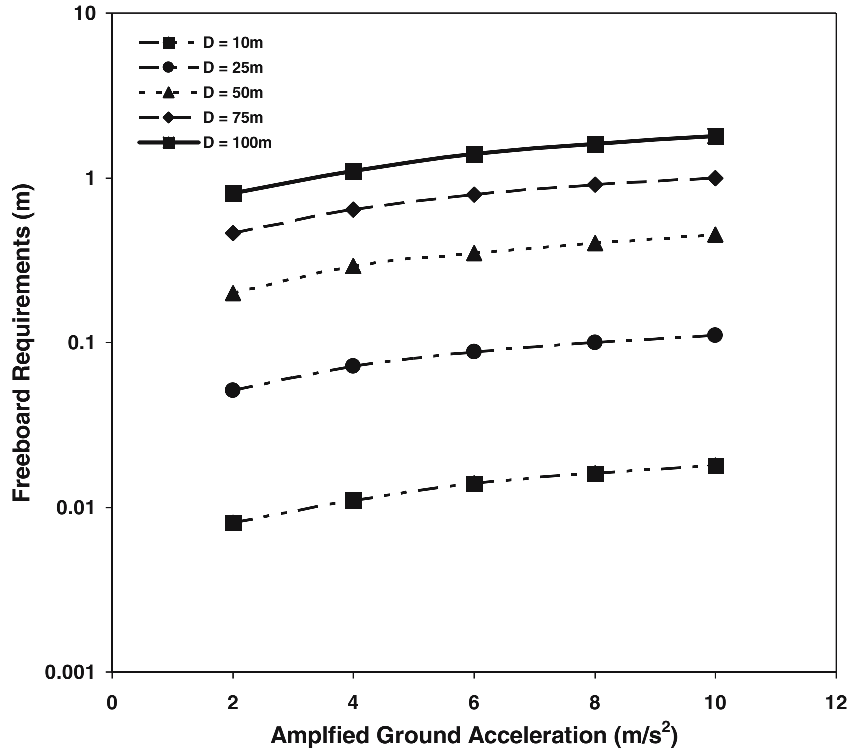


Fig. 5 Freeboard requirements of surface impoundment in soft soil under seismic action (impoundment slope is 30°)

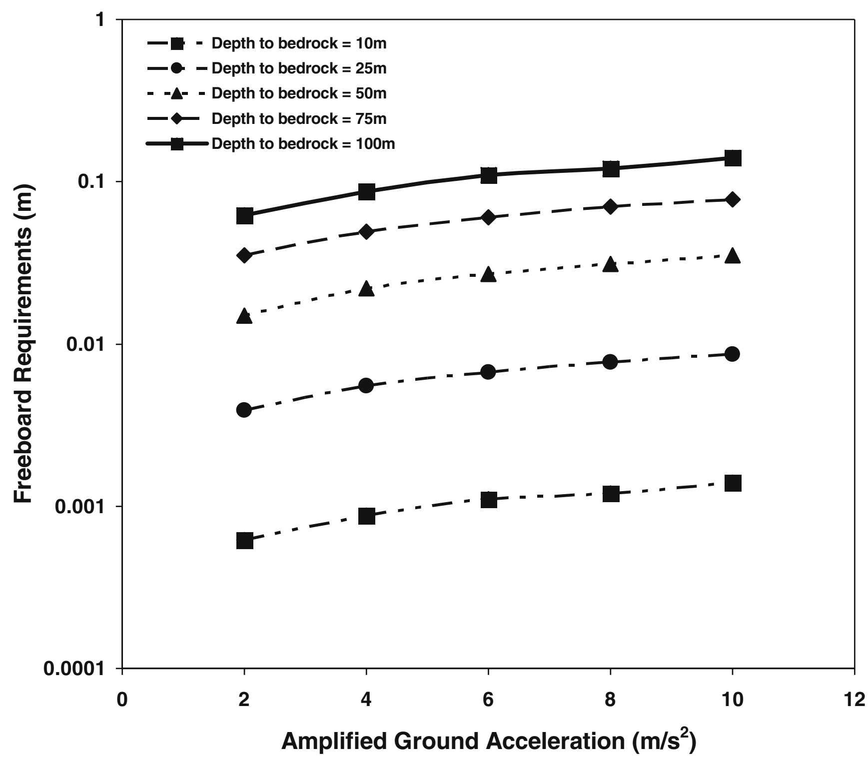


Fig. 6 Freeboard requirements of surface impoundment in medium-dense soil under seismic action (impoundment slope is 10°)

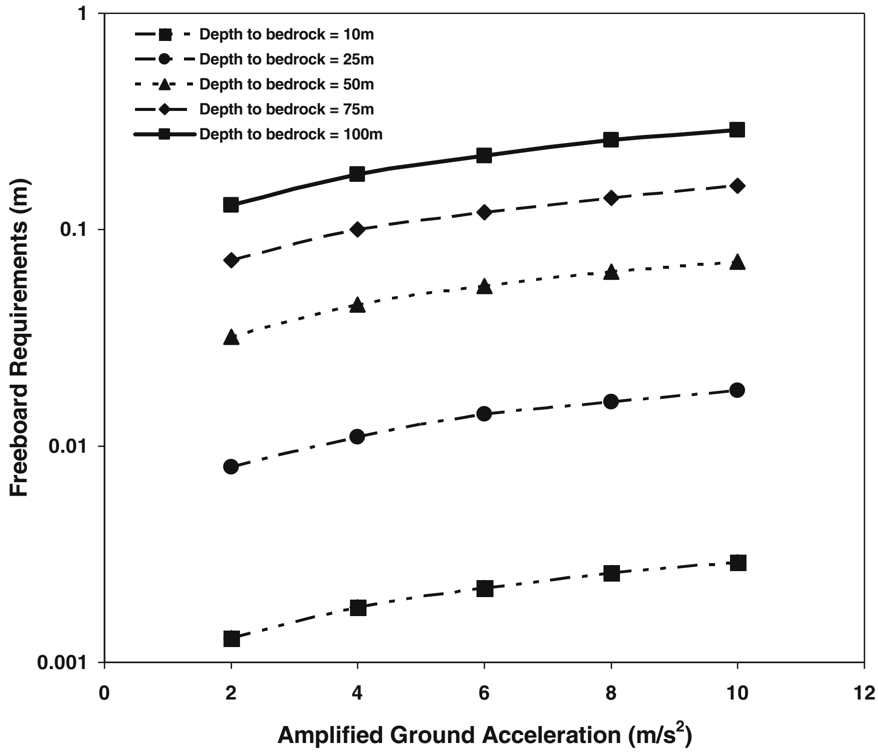


Fig. 7 Freeboard requirements of surface impoundment in medium-dense soil under seismic action (impoundment slope is 20°)

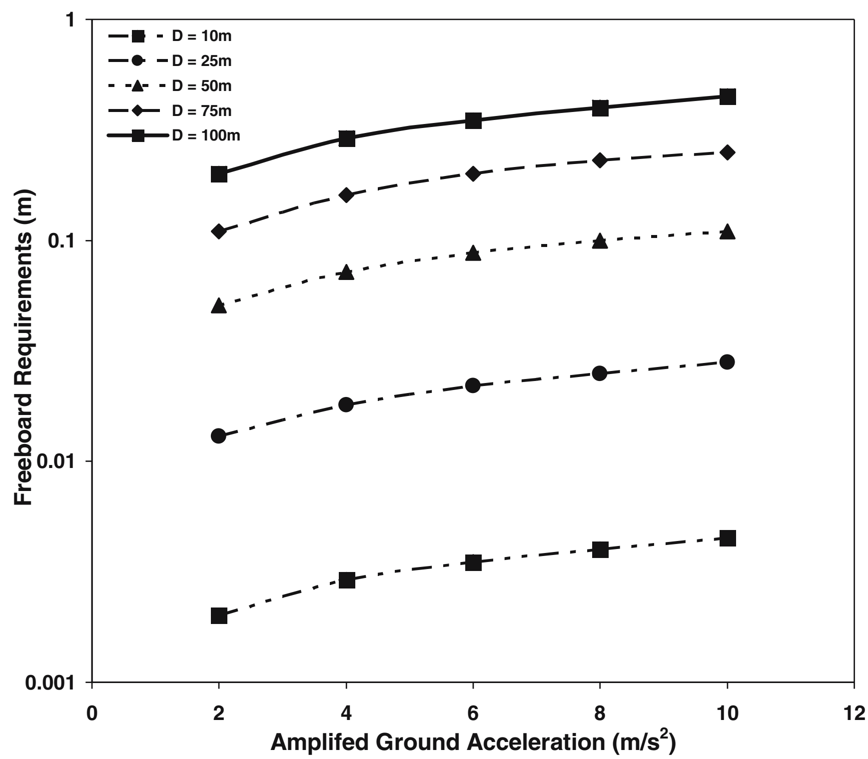


Fig. 8 Freeboard requirements of surface impoundment in medium-dense soil under seismic action (impoundment slope is 30°)

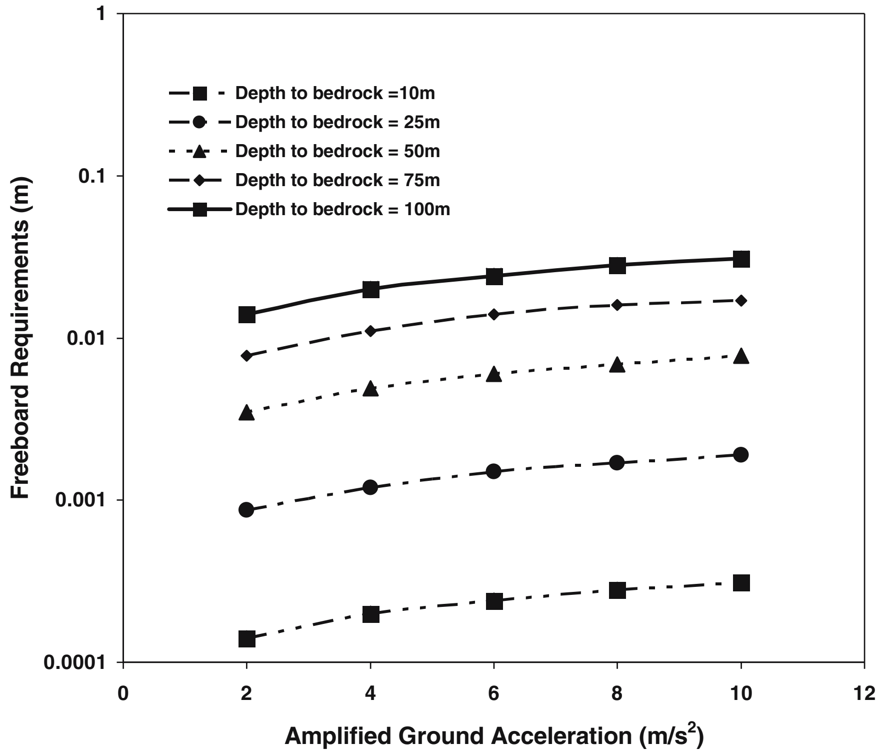


Fig. 9 Freeboard requirements of surface impoundment in dense soil under seismic action (impoundment slope is 10°)

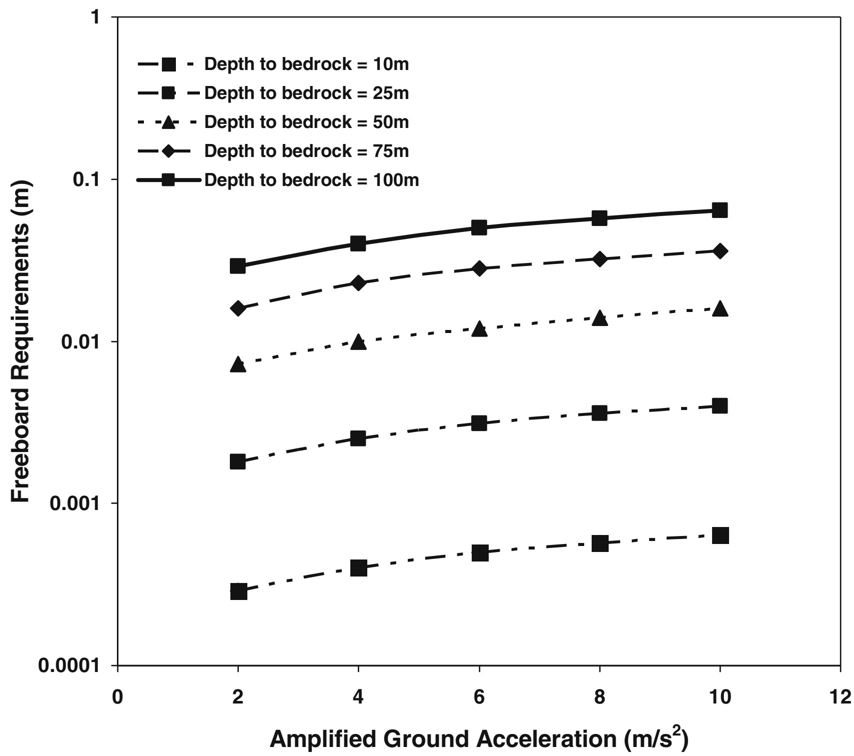


Fig. 10 Freeboard requirements of surface impoundment in dense soil under seismic action (impoundment slope is 20°)

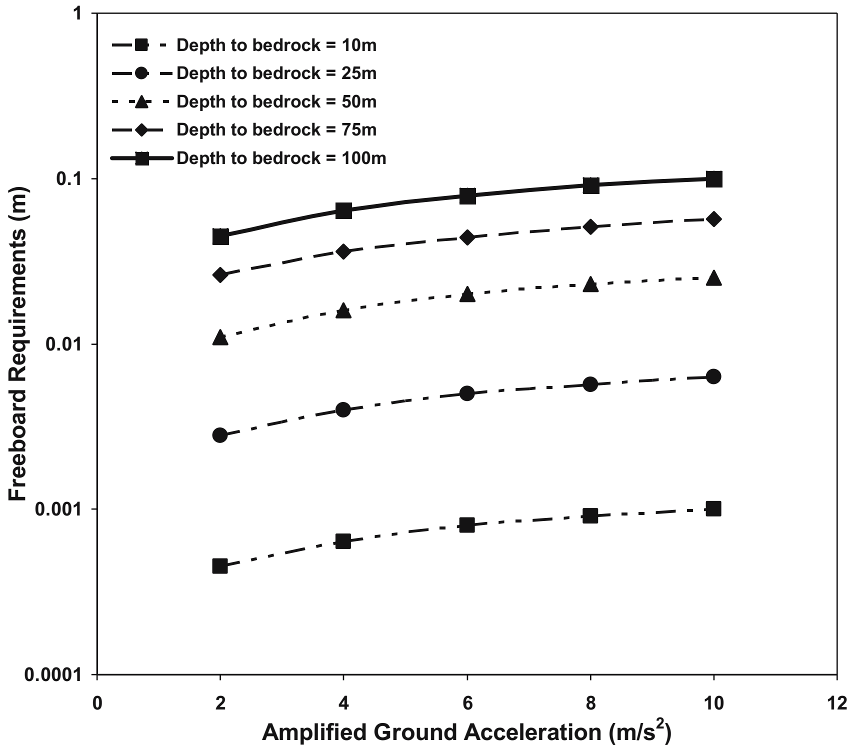


Fig. 11 Freeboard requirements of surface impoundment in dense soil under seismic action (impoundment slope is 30°)

Numerical example: estimation of freeboard requirements against seismic-induced overtopping of surface impoundments in three different regions of the US

To demonstrate the application of the formulated expression, consider a surface impoundment with slope of 15° and average impounded liquid depth of 5 m in a region with the following geological and seismic characteristics: 10% or greater probability that the maximum horizontal ground acceleration (PGA) will exceed 1.0 g m/s^2 in 250 years; and shear wave velocity, site amplification factors, and depth to bedrock are 180 m/s, 1.5, and 50 m, respectively.

With reference to Eq. 33, $Z = 5\ m$, $V_s = 180\ m/s$, $D = 50\ m$, $a = 1.0\ g\ m/s^2$, and $\alpha = 15°$. The freeboard requirement under this scenario is calculated as follows using Eq. 33:

$$F = \frac{8 \times 50^2 \tan 15°}{\pi^2 \times 180^3}\left\{9.8 \times \left[1.5 \times 10 \times 180^2 \times \pi^2 \times 5 \cos\left(\frac{\pi \times 5}{2 \times 50}\right) - 50^2 \times (1.5 \times 10)^2 \times \cos^2\left(\frac{\pi \times 5}{2 \times 50}\right)\right] \Big/ 2\pi \times 5\right\}^{1/2} = 0.25\,\mathrm{m}.$$

Conclusion

A mathematical expression that is based on spatially distributed parameters has been developed for quantifying magnitude of freeboard required against seismic-induced overtopping of surface impoundments. The derivation can easily be incorporated into human and ecological health risk models by computing the magnitude of contaminant source term concentration under seismic-induced overtopping of impoundments.

The performance of impoundments in terms of resistance to liquid overtopping has been shown to be dependent on impoundment design in terms of side slope and height; site characteristics, primarily depth and density of underlying soil; and characteristics of incident wave and ground acceleration. A quantitative relationship has been developed by extending existing models that describe the response of fluids to underlying bed excitation by seismic waves.

Application of the derived model to typical and bounding scenarios indicates that the freeboard size required to prevent overtopping ranges from about 0.0002 to about 2.0 m for soil depths of 10–100 m over bedrock subjected to ground accelerations of 0.2–1.0 g by seismic shear waves traveling at a velocity of 180 m/s. The thickness and density of the soil layer between the bottom of the impoundment and the seismically impacted bedrock is a very significant parameter with respect to the size of waves that develop in the impounded liquid. High amplitude waves require larger freeboard. Shallower soil conditions above an excited bedrock produce larger waves in the liquid, but the effects of soil thickness on required freeboard decreases as the soil thickness increases. As expected, the required freeboard is also directly proportional to the impoundment slope angle.

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