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Analysis of earth dams affected by the 2001 Bhuj Earthquake

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Abstract

An earthquake of magnitude of 7.6 (M_w 7.6) occurred in Bhuj, India on January 26, 2001. This event inflicted damages of varying extents to a large number of small to moderate size multi-zone earth dams in the vicinity of the epicenter. Some of the distress was due to the liquefaction of saturated alluvium in foundation. Liquefaction was relatively localized for the majority of these dams because the earthquake struck in the middle of a prolonged dry season when the reservoirs behind these dams were nearly empty and shallow alluvium soils underneath the downstream portions of the dams were partly dry. Otherwise, liquefaction of foundation soils would have been more extensive and damage to these dams more significant. Six such dams have been examined in this paper. Four of these facilities, Chang, Shivlakha, Suvi, and Tapar were within the 50 km of epicenter region. These dams underwent free-field ground motion with peak ground accelerations between 0.28g to 0.52g. Of these Chang Dam underwent severe slumping, whereas Shivlakha, Suvi, and Tapar Dams were affected severely especially over the upstream sections. Fatehgadh Dam and Kaswati Dam were affected relatively less severely. Foundation conditions underneath these dams were first examined for assessing liquefaction potential. A limited amount of subsurface information available from investigations undertaken prior to the earthquake indicates that, although the foundation soils within the top 2.0 to 2.5 m underneath these dams were susceptible to liquefaction, Bhuj Earthquake did not trigger liquefaction because of lack of saturation of these layers underneath the downstream portions of these dams. These dams were then analyzed using a simple sliding block procedure using appropriate estimates of undrained soil strength parameters. The results of this analysis for these structures were found to be in general agreement with the observed deformation patterns.

Keywords: Earthquake; Liquefaction; Embankment dam; Pseudo-static; Sliding block; Deformation; Slope stability

1. Introduction

An earthquake of magnitude of 7.6 (M_w 7.6) occurred on January 26, 2001. The epicenter of the main shock of the event was located near Bachau at latitude

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23.36°N and longitude 70.34°E with a focal depth of about 23.6 km. The event, commonly referred to as the Bhuj Earthquake, was among the most disastrous earthquakes that have affected India.

Bhuj Earthquake affected a large number of small- to moderate-size water-retaining earthen dams and reservoirs, constructed to fulfill the water demand of the area. Most of these dams are embankment dams typically constructed across discontinuous ephemeral streams. Although a number of such structures were within 150 km of the epicenter (Fig. 1), the consequences of the damage caused by the earthquake to these dams and ancillary structures were relatively light. This is primarily because of the low reservoir levels during the earthquake. The nature of damage to the embankment dams within the epicentral region is summarized in Table 1.

The performance of six embankment dams affected by Bhuj Earthquake is examined here. Among these, Chang Dam underwent almost a complete collapse because of liquefaction of shallow foundation soils. Shivlakha Dam was also severely damaged leading the failure of the upstream slope presumably because of liquefaction underneath the upstream portion of the dam. Damages to Suvi, Tapar, Fatehgadh, and Kaswati Dams were relatively less severe and confined near the upstream toe, upstream slope, and dam crest.

Limited subsurface data available from investigations prior to Bhuj Earthquake were analyzed using the simplified procedure for assessment of liquefaction potential (Youd et al. 2001). These analyses indicate a likelihood of widespread liquefaction of shallow alluvium soils underneath Chang Dam, while for Shivlakha, Tapar, Suvi, Fatehgadh, and

Table 1
Observed performance of selected dams

Dam	Crest length, height (m)	a_{max}	R (km)	Distress
Chang	370, 15.5	0.50g	13	Liquefaction in foundation, failure of upstream and downstream slopes, slumping, cracking
Shivlakha	300, 18.0	0.45g	28	Possible liquefaction in foundation, upstream and downstream slope failure, cracking
Fatehgadh	4049, 11.6	0.30g	80	Possible liquefaction in foundation near upstream toe, shallow failure in upstream slope, cracking
Kaswati	1455, 12.9	0.28g	110	Possible liquefaction in foundation near upstream toe, shallow failure in upstream slope, cracking, leakage
Suvi	2097, 15.0	0.42g	37	Possible liquefaction in foundation near upstream toe, shallow failure in upstream slope, cracking
Tapar	1350, 15.5	0.41g	43	Liquefaction in foundation near upstream toe, shallow failure in upstream slope, cracking

Notes: (1) estimates for a_{max} are based on Singh et al. (2003) attenuation relationship and Idriss (1990) site amplification relationship. (2) R is the approximate epicentral distance.

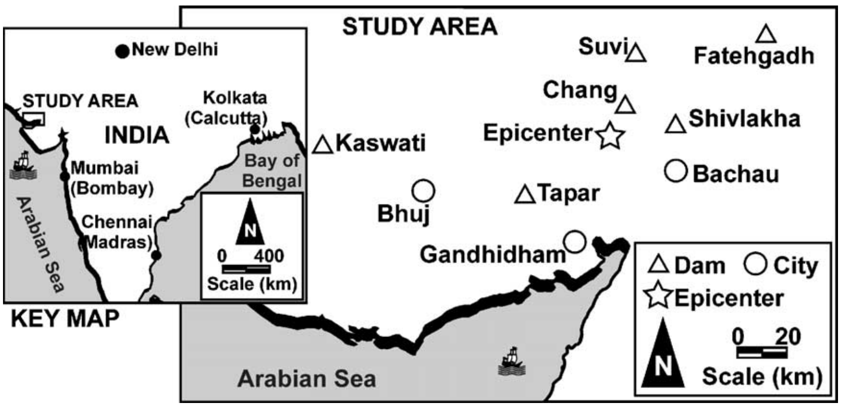

Fig. 1. Study area.

Kaswati Dams liquefaction could only have occurred underneath the upstream slope because the foundation soils were partially dry at the time of the earthquake or because of overburden pressure due to the dam structure.

The dams were subsequently analyzed using the sliding block method originally developed by Newmark (1965) and the design charts developed by Hynes-Griffin and Franklin (1984) facilitating the use of the Newmark procedure for estimating the magnitude of deformation. The deformations estimated from the sliding block procedure were compared with the observed deformation patterns following the Bhuj Earthquake to check the predictive capability of this simple procedure. The results indicate a reasonable agreement between the deformations estimated from the sliding block procedure and observed distress pattern. However, it should be noted that these analyses are based on limited sub-surface data from investigations undertaken before the occurrence of Bhuj Earthquake and ground motion estimates in a setting where site-specific earthquake records are not available.

2. Observed dam performance

A brief summary of the performance of the six dams examined in this study is provided in the following subsections. For a more detailed account of the post-earthquake damage survey at dam sites reference may be made to the EERI (2001) Reconnaissance Report.

2.1. Chang Dam

Chang Dam, constructed in 1959, is a multi-zone earth dam with 15.5 m height at its maximum section and 370 m crest length (Fig. 2). The dam is founded on sand and silt mixtures over shallow sandstone bedrock. Liquefaction susceptibility of the foundation soils was not studied or considered in the original design. Although the reservoir behind Chang Dam was nearly empty at the time of Bhuj Earthquake, the alluvium soils underneath the dam were probably in a saturated state during the earthquake. EERI (2001) reports a significant distress within the dam body including the impervious core and the masonry wall as a result of Bhuj Earthquake. Sand boils were observed near the upstream toe of Chang Dam following the earthquake. The observed deformation pattern (Fig. 2) is also indicative of widespread liquefaction within the foundation soils.

2.2. Shivlakha Dam

Shivlakha Dam, constructed in 1954, is a multi-zone earth dam with 18.0 m height at its maximum section and 300 m crest length (Fig. 3). The site is underlain by sand and silt mixtures over shallow bedrock. Although the reservoir behind Shivlakha Dam was nearly empty at the time of Bhuj Earthquake, the alluvium underneath the dam appears to have been in a saturated state. Liquefaction underneath the upstream shell triggered by Bhuj Earthquake led to the failure of the upstream slope and develop-

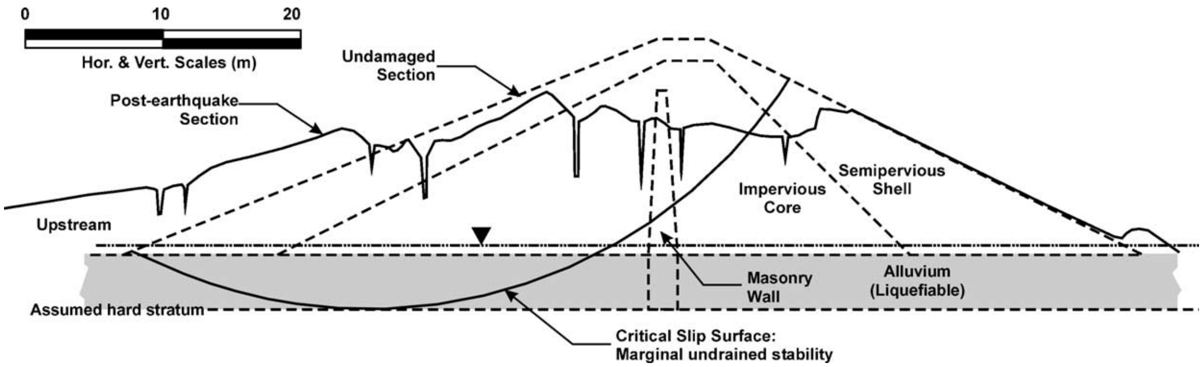

Fig. 2. Cross-section of Chang Dam.

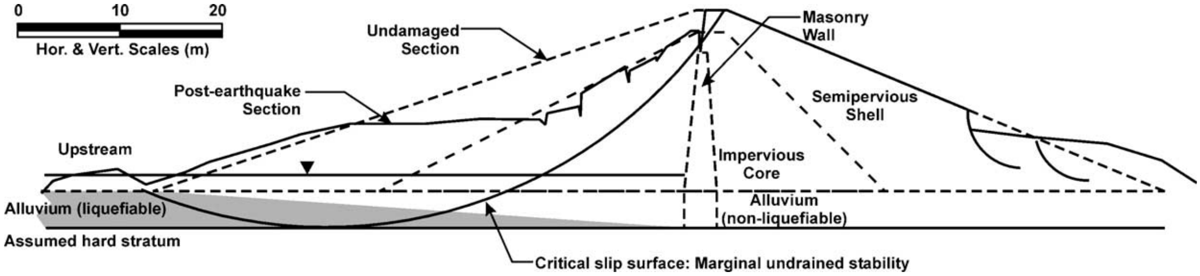

Fig. 3. Cross-section of Shivlakha Dam.

ment of large fissures near the upstream toe. The deformed shape of the dam section is presented in Fig. 3 together with its pre-earthquake configuration for comparison.

2.3. Tapar Dam

Tapar Dam, constructed in 1976, is a multi-zone earth dam with 15.5 m height at its maximum section and 1350 m crest length (Fig. 4). It was raised by an additional 2.5 m in the 1990s. The Dam is founded directly upon alluvium soils to a depth of greater than 30 m. Although, Tapar Reservoir was nearly empty at the time of Bhuj Earthquake, the alluvium underneath the upstream portion of the dam was in a saturated state. Bhuj Earthquake caused a significant distress to the dam especially within the upstream portion (EERI 2001). Sand boils were observed near the upstream toe of Tapar Dam following the earthquake. Liquefaction beneath the upstream toe caused lateral spreading and translational movements of several sections of the upstream slope. The deformed shape of the dam section is included in Fig. 4 together with its pre-earthquake configuration for comparison.

2.4. Fatehgadh Dam

Fatehgadh Dam, constructed in 1979, is a multi-zone earth dam with a maximum height of 11.6 m and crest length of 4050 m (Fig. 5). Like Chang Dam, Fatehgadh Dam is also underlain by loose to medium dense silt sand mixtures. Limited amount of subsurface exploration data indicate that the site is underlain by 2 to 5 m thick granular soils characterized with an uncorrected Standard Penetration Test (SPT) blow count between 13 and 19.

During Bhuj Earthquake the reservoir level was near ground surface and the alluvium underneath the dam was in a saturated state. The earthquake triggered shallow sliding especially near the bottom portion of upstream slope (EERI, 2001). Such distress could be caused by localized liquefaction near the upstream toe of the dam. The EERI reconnaissance team also reports development of cracks as deep as 1.5 to 1.7 m within the upstream portion of the dam and instability near the top portion of the downstream slope following the earthquake. The problem of appearance of longitudinal cracks may also indirectly relate to liquefaction of foundation soils. However, instability of the upper portion of the downstream slope may not be due to the lique-

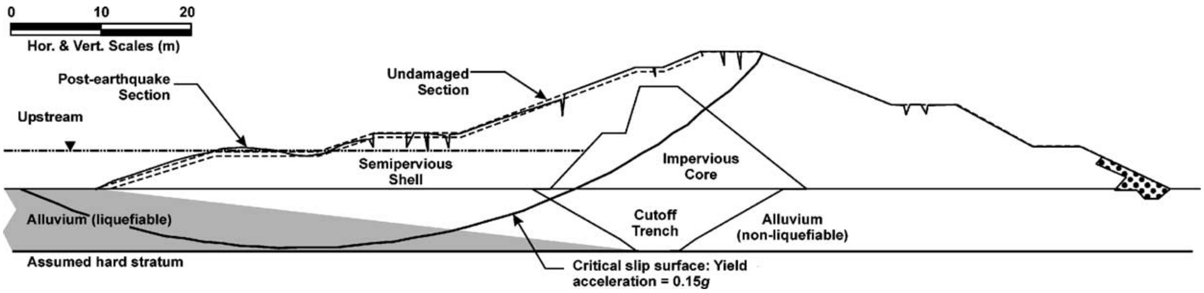

Fig. 4. Cross-section of Tapar Dam.

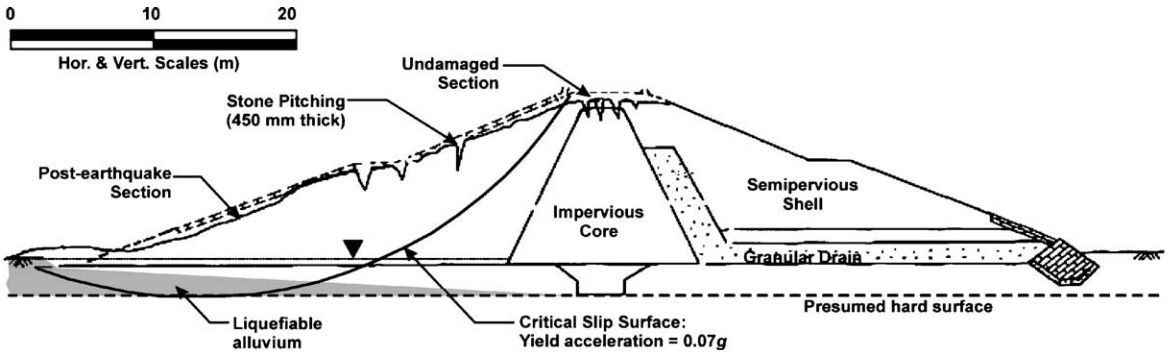

Fig. 5. Cross-section of Fatehgadh Dam.

faction of foundation soils. The deformed shape of the dam section is included in Fig. 5 together with its pre-earthquake configuration for comparison.

2.5. Kaswati Dam

Kaswati Dam, constructed in 1973, is a multi-zone earth dam with a maximum height of 12.9 m and crest length of 1455 m (Fig. 6). The dam is underlain by loose to medium dense silt sand mixtures. Limited amounts of subsurface exploration data indicate that the site is underlain by 2 to 5 m thick granular soils characterized by an uncorrected SPT blow count between 13 and 19.

Like the other impoundments considered in this study, the reservoir behind Kaswati Dam was nearly empty at the time of Bhuj Earthquake but the alluvium soil underneath the dam was in a saturated state. EERI (2001) report triggering of shallow sliding, especially near the bottom portion of upstream slope, and bulging of ground surface near the upstream toe as a result of Bhuj Earthquake. Such distress may have been caused by localized liquefaction near the upstream toe of the dam. The EERI reconnaissance team report development of relatively narrow, longitudinal cracks along the crest of the dam running the length of the dam over which the lower portion of the upstream slope exhibited distress. It appears therefore the problem of development of longitudinal cracks along the crest is indirectly related to liquefaction of foundation soils. The downstream slope, on the other hand, remained largely unaffected. The deformed shape of the dam section is included in Fig. 6 together with its pre-earthquake configuration for comparison.

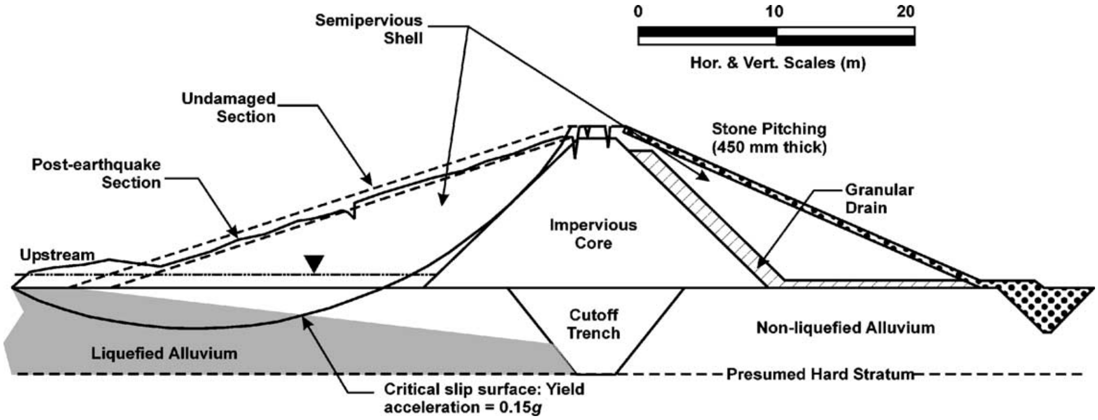

Fig. 6. Cross-section of Kaswati Dam.

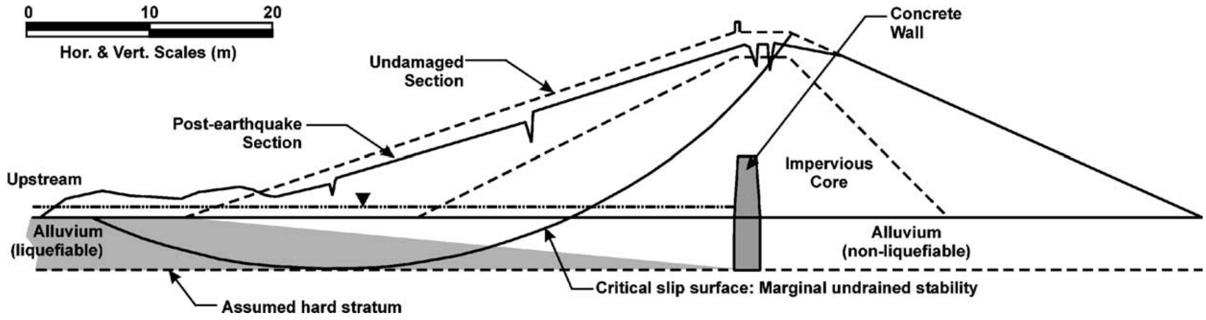

Fig. 7. Cross-section of Suvi Dam.

2.6. Suvi Dam

Suvi Dam, constructed in 1959, is a multi-zone earth dam with 16.5 m height at its maximum section and 2100 m crest length (Fig. 7). It was raised by an additional 1.0 m in the 1990s. The dam is underlain by loose to medium dense silt sand mixtures.

Like the other impoundments considered in this study, the reservoir behind Kaswati Dam was nearly empty at the time of Bhuj Earthquake. Based on the presence of vegetation along the downstream portion of the dam, Krinitzsky and Hynes (2002) inferred ongoing seepage through the shallow alluvium foundation soils underneath Suvi Dam. It appears therefore that during Bhuj Earthquake, the alluvium soil underneath the dam was in a saturated state. EERI (2001) report a crest parapet wall of stone masonry was demolished by inertial force along 60% of the crest length. The upstream slope failure was similar to those at Fatehgadh and Kaswati Dams observed following the Bhuj Earthquake. In addition, the crest of Suvi Dam subsided by up to 1 m along approximately a 200-m long segment. Open Fissures were observed in upstream face of the Dam.

3. Assessment of liquefaction potential

Essential details of the procedure for assessing liquefaction potential and the results of this assessment are presented in the following subsections.

3.1. The SPT-based procedure

The procedure for assessing liquefaction potential typically uses the Cyclic Resistance Ratio (*CRR*) as a measure of the liquefaction resistance of soils and the Critical Stress Ratio (*CSR*) as a measure of earthquake load. For cohesionless soils, *CRR* has been related to normalized SPT blow count, $(N_1)_{60}$, through correlations that depend on the fines content of the soil from field performance observations from past earthquakes (e.g., Fig. 8). The normalized SPT blow count is given by:

$$(N_1)_{60} = N \times (P_a/\sigma'_{v0})^{0.5} \times ER \quad (1)$$

where N is the raw SPT blow count, P_a is the atmospheric pressure ($\approx$ 100 kPa), σ'_{v0} is the effective vertical stress at the depth of testing, and ER is the energy ratio ($\approx$ 0.92 in a typical Indian SPT setup).

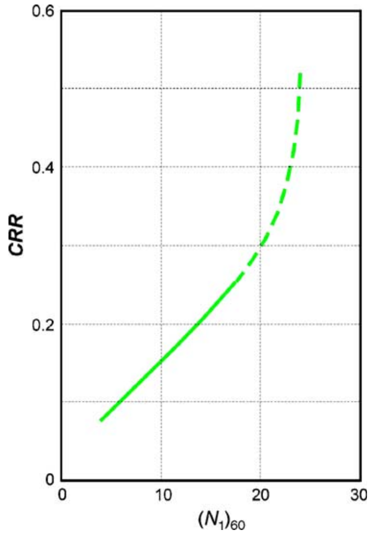

Fig. 8. CRR–$(N_1)_{60}$ correlation for soils with fines content between 5% and 15% (modified from Youd et al. 2001).

The procedure for assessing liquefaction potential uses the CSR as the measure for earthquake load, where

$$CSR = 0.65 \times (a_{\max}/g) \times \left(\sigma_{v0}/\sigma'_{v0}\right) \times r_d \times K_m^{-1} \times K_\alpha^{-1} \times K_\sigma^{-1} \quad (2)$$

where $a_{\max}$ is the peak horizontal ground acceleration, g is the acceleration due to gravity, σ_{vo} is the total vertical stress, r_d is a correction factor to account for the flexibility of the soil column, and K_m, K_α and K_σ are correction factors to account for the Magnitude of the earthquake, the presence of initial static shear (i.e., whether the layers are in a slope) and the depth of the layer (i.e., the level of initial overburden pressure), respectively. We estimated the value of r_d for a given depth from Seed et al. (2003) median relationship. Correction factors K_m, K_α and K_σ were obtained from the relationships recommended by Youd et al. (2001) using estimates of relative density obtained from (Olson and Stark, 2003b):

$$D_r = \sqrt{(N_1)_{60}/44} \quad (3)$$

3.2. Sub-surface conditions

The limited amount of SPT data available from Fatehgadh and Kaswati Dams indicates that the shallow foundation soils underneath the dam body were characterized by blow counts between 13 and 19. For assessing liquefaction potential of foundation soils we assumed that the fines content of these shallow alluvium layers were 15% or less.

3.3. Assessed liquefaction potential

The results of assessment of liquefaction susceptibility of the foundation soils underneath Chang, Shivlakha, Tapar, Fatehgadh, Kaswati, and Suvi Dams are presented in Table 2. These results indicate that the foundation soils are liquefiable for the estimates of horizontal peak horizontal ground accelerations at dam sites (see Table 1 for a listing) under free-field conditions. This assessment is in agreement with the observed or inferred liquefaction near the upstream toes of Shivlakha, Tapar, Fatehgadh, Kaswati, and Suvi Dams. Since the shallow foundation soils near the downstream toes of these dams were partially saturated, liquefaction was not triggered in the vicinity of downstream toes. This inference is in agreement with the lack of direct evidence of liquefaction downstream of the dams and the fact that the downstream slopes of the dams generally performed better than the upstream slopes.

Table 2
Liquefaction susceptibility of foundation soils

Dam	CRR		CSR		Liquefaction susceptibility	
	Crest	Toe	Crest	Toe	Crest	Toe
Chang	0.32	0.32	0.35	0.73	Yes	Yes
Shivlakha	0.32	0.32	0.31	0.63	Marginal	Yes
Tapar	0.32	0.32	0.30	0.62	Marginal	Yes
Fatehgadh	0.32	0.32	0.18	0.35	No	Yes
Kaswati	0.32	0.32	0.17	0.34	No	Yes
Suvi	0.32	0.32	0.31	0.63	No	Yes

The semi-pervious shell and impervious core of the multi-zone earth dams studied in this research are compacted, cohesive, and partially saturated. The drainage filter is non-cohesive but partially saturated. Such soils are not susceptible to liquefaction.

Foundation soils downstream of the crest of the dam were also partially saturated at the time of the earthquake. These soils were therefore also considered non-liquefiable. Estimated extent of liquefied soils underneath Chang, Shivlakha, Tapar, Fatehgadh, Kaswati and Suvi Dams are shown in Figs. 2–7, respectively.

4. The sliding block method

The sliding block method was used in this study to estimate the deformation potentials for Chang, Shivlakha, Tapar, Fatehgadh, Kaswati, and Suvi Dams. Procedural details and results from these analyses are as follows.

4.1. Procedural details

In this method, the potential sliding mass is approximated as a rigid body resting on a rigid sloping base and the contact between the potential sliding mass and the underlying slope is assumed as rigid-plastic (Newmark, 1965). The potential sliding mass would move down slope relative to the sloping base when the down slope ground acceleration exceeds a threshold value required to overcome the cohesive-frictional resistance at the contact between the sliding mass and the rigid base. For a single pulse of down slope earthquake acceleration that sets the potential sliding mass into motion, the instantaneous velocity of the sliding mass relative to the sloping base is obtained by integrating the amount by which the earthquake acceleration exceeds the mobility threshold with respect to time. When the magnitude of down slope earthquake acceleration drops back below the mobility threshold, the sliding mass would decelerate because of cohesive-frictional energy loss before losing mobility relative to the base. To obtain the magnitude of incremental, relative, down slope displacement of the sliding mass for the earthquake acceleration pulse considered above, the instantaneous relative velocity is integrated against time. The total, relative, down slope displacement of the sliding mass is then estimated by summing up all such incremental relative displacements over the entire duration of the earthquake ground motion. The displacements are considered irreversible, i.e., only the down slope component of acceleration is considered and the effects of the up slope acceleration pulses are neglected.

The threshold acceleration above which down slope movement of the potential failure mass is triggered is referred to as the yield acceleration, a_y. To estimate the yield acceleration for a given slope geometry, a limit equilibrium slope stability analysis is undertaken. The inertial effect due to the design earthquake is included in the analysis typically in the form of a horizontal seismic coefficient. The horizontal seismic coefficient is multiplied by the weight of the potential sliding block, i.e., the volume of soil above the trial sliding surface and below slope surface to obtain a crude estimate of the inertial effect of the earthquake. Such an analysis is sometimes referred to as pseudo-static slope stability analysis. Therefore, the horizontal seismic coefficient represents the average amplitude of earthquake ground motion within the potential sliding mass. Yield acceleration is assumed to be equal to the horizontal seismic coefficient that gives a limit equilibrium factor of safety of unity. The influence of the vertical component of earthquake-related ground motion has not been considered in this study.

As is apparent from the preceding discussion, the procedure does not include the flexibility of a potential sliding mass because of which the entire volume of soils above the trial failure surface may not be mobilized in the same direction simultaneously. This limitation is especially important for earthquakes with higher predominant frequency. The error that results because of this limitation usually leads to an overestimation of permanent deformation. However, since the foundation soils underneath the dams examined in this study liquefied to various extents during Bhuj Earthquake, the higher frequencies of the ground motion may have been filtered out partially because of increased damping that results from liquefaction. Secondly, the assumption that the contact between the potential sliding block and the sloping base is rigid plastic is not a good representation of material behavior especially for soils that are susceptible to significant softening, e.g., soils susceptible to liquefaction and sensitive cohesive deposits. To accommodate conservatively the triggering of liquefaction within the foundation soils during Bhuj Earthquake within the sliding block framework, the post-liquefaction shear strength for liquefied soils has been used in the pseudo-static slope stability analysis.

To facilitate the use of the Newmark (1965) framework, Hynes-Griffin and Franklin (1984) related the estimated deformations of a sliding block to the ratio of yield acceleration to peak horizontal ground acceleration at the elevation of the toe of the dam through upper-bound, median, and lower-bound correlations. The upper-bound relationship proposed by Hynes-Griffin and Franklin (Fig. 9) has been used in this study. The Hynes-Griffin and Franklin chart is useful in situations where the Newmark (1965) procedure cannot be used directly because of the difficulty in selecting a suite of design earthquake acceleration

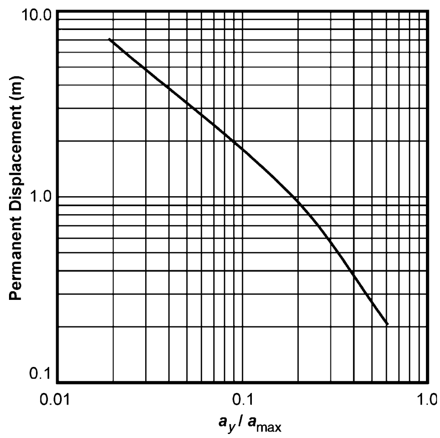

Fig. 9. The upper-bound correlation between a_y/a_{max} and permanent deformation (modified from Hynes-Griffin and Franklin, 1984).

time histories that represent the seismo-tectonic and geologic settings or because of a dearth of available earthquake records as was the case with the study area of this research.

5. Analyses and results

Computer program XSTABL version 5.2 (Interactive Software Designs, Inc., 1994) and the Modified Bishop method were used in the pseudo-static slope stability analyses. The input parameters used in the analyses are listed in Table 3. For the semi-pervious shell within dam body, the assumed soil properties of Table 3 reflect typical shear strengths of materials used in Dam construction in the study area (Nadpura and Ramchand, 2005). The strength parameters of the liquefied and non-liquefied portions of the foundation alluvium layers, on the other hand, were obtained from Olson and Stark (2003a).

The critical slip surfaces obtained in the analyses are superposed in Figs. 2–7 and the yield accelerations are listed in Table 4. These results indicate that the yield accelerations for Chang, Shivlakha and Suvi Dams under undrained loading conditions are less than 0.05g. These dams are thus only marginally stable under undrained earthquake loading. The yield acceleration for Fatehgadh Dam is 0.07g, while that for Tapar and Kaswati Dams is 0.15g.

The yield accelerations were then used along with the hard soil ground motion estimates at dam sites obtained from Singh et al. (2003), site amplification from Idriss (1990), and the upper-bound correlations developed by Hynes-Griffin and Franklin (1984) to estimate permanent deformation (see Fig. 9). These results are summarized in Table 4. The results presented in Figs. 2–7 and Table 4 are in reasonable agreement with post-earthquake observations.

Table 3
Soil properties used in undrained limit equilibrium slope stability assessment

Soil unit	Unit weight (kN/m^3)	Cohesion (kPa)	ϕ	s_u/σ'_v
Semi-pervious shell	18	9	30°	
Impervious core	20	65	0	
Masonry wall	22	80	0	
Liquefied foundation soil	18	0.0		0.195
Non-liquefied foundation soil	18	0.0		0.370
Deep alluvium	20	0.0	41°	

Table 4
Yield accelerations, and estimated and observed displacements

Dam	Yield acceleration	Estimated displacement (m)	Observed displacement	
			Horizontal (m)	Vertical (m)
Chang	Marginal undrained stability	> 8	7.1	4.3
Shivlakha	Marginal undrained stability	1.2	1.2	2.0
Tapar	0.15g	0.4	0.6	0.5
Fatehgadh	0.07g	0.7	0.6	0.6
Kaswati	0.15g	0.3	0.6	0.5
Suvi	Marginal undrained stability	2.0	1.2	2.0

6. Conclusions

A simple method of analysis has been used to estimate the permanent deformations within six earth dams due to Bhuj Earthquake. These dams partially fulfill the irrigation and drinking water needs of a semi-arid area that was affected by the M_w 7.6 earthquake. Although these facilities were within 150 km from the epicenter of the earthquake, only one of the three dams collapsed because of the earthquake. The performance could, however, have been worse had the reservoirs been full when the earthquake occurred.

An approximate but simple analytical procedure has been used to estimate the permanent deformation of the dams. The procedure is based on the upper-bound relationship between the ratio a_y/a_{max} and permanent deformation developed by Hynes-Griffin and Franklin (1984). The results of these analyses are in agreement with the pattern of permanent deformation of the earth dams observed following the earthquake. It appears therefore that the simple yet relatively inexpensive tool has a potential for making quick assessments of seismic safety of similar earth dams designed and constructed without considering earthquake loading.

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