American J. of Engineering and Applied Sciences 2 (3): 515-525, 2009
ISSN 1941-7020

Estimation of Earthquake-Induced Crest Settlements of Embankments

Raghvendra Singh and Debasis Roy
Department of Civil Engineering, Indian Institute of Technology Kharagpur, WB 721302, India

Abstract: Problem statement: Freeboard requirement, a major consideration in the design of embankment dams, is controlled to a great extent by the crest settlements during earthquakes. **Approach:** The parameters that influence earthquake induced crest settlement had been studied using 152 published case histories on performance of embankment dams during earthquakes. **Results:** Based on the results a correlation had been proposed for obtaining preliminary estimates of earthquake-induced crest settlements. The correlation used the ratio of the peak horizontal ground acceleration and the yield acceleration as the estimator. **Conclusion**/**Recommendations:** The database analysis also indicated that crest settlements are larger where the fundamental periods of the embankment were similar to the predominant periods of the earthquake. Earthquake magnitude and the vertical component of earthquake ground motion, on the other hand, appeared to have a small influence on crest settlement.

Key words: Peak horizontal ground acceleration, dams, embankments, earthquake, fundamental period, predominant period

INTRODUCTION

Earthquake-induced deformations of embankment dams are commonly estimated from the sliding block procedure[35]. In this procedure the yield acceleration, a_y, or the horizontal seismic coefficient that gives a pseudo static limit equilibrium factor of safety of unity against slope failure is first estimated from pseudo static slope stability analyses. The pseudo static slope stability analysis is essentially an extension of conventional, static stability analysis in that it approximately accounts for the inertial effects by using inertial forces in proportion of horizontal and vertical seismic coefficients depending on the ground acceleration levels representing the design earthquake. In the sliding block approach the permanent deformation is estimated for a particular earthquake acceleration time history by identifying the time intervals over which the down slope earthquake acceleration exceeds a_y and doubly integrating the difference between the down slope earthquake acceleration and a_y over these intervals considering rigid plastic material behavior and inertia-related overshooting. A slope is usually assumed to be stable during earthquakes if the estimated down slope deformation is smaller than 1 m.

As in pseudo static method, the sliding block method assumes the sliding wedge (the volume of soil above the slip surface) to be rigid plastic whereas in reality the soil is flexible. Also contrary to the assumption of these methods that the cohesive-frictional strength between the sliding wedge and the soil mass below do not degrade with continued earthquake shaking, loose or soft soils tend to soften during earthquakes because of development of pore water pressure. Another difficulty with the use of sliding block method is the need for a suite of design earthquake acceleration time histories, which, in many cases, is not available. Nevertheless because of the simplicity of these methods, they often are the procedures of choice in a preliminary design project.

Semi empirical or performance-based charts have been developed, e.g., Singh *et al*.[49] for obtaining preliminary estimates of permanent, earthquake-induced deformation of earth dams and embankments from the results of limit-equilibrium, pseudo-static slope stability assessment in situations where site specific design earthquake acceleration time histories are unavailable[17,26,45]. The procedure developed by the Hynes-Griffin and Franklin[17], for instance, extends the Newmark[35] sliding block method relating the deformation of dam section to the ratio of yield acceleration, a_y and the peak horizontal ground acceleration at dam base, a_{max} based on stability assessments of slopes for a large number of natural and synthetic earthquake acceleration time

Corresponding Author: Debasis Roy, Department of Civil Engineering, Indian Institute of Technology Kharagpur, WB 721302, India Tel: 91 3222 283456/91 9333 451843 Fax: 91 3222 282254

histories. Singh *et al*.[49], on the other hand, related permanent down slope deformation to a_y/a_{max} based on dam performance observations from past earthquakes.

Unfortunately, these procedures based on the sliding block approach are generally silent about the physical significance of the representative permanent down slope displacements. Assuming the down slope displacement to align with the average inclination of the base of the critical slip surface, crest settlement could however be roughly estimated from the down slope displacement obtained from the sliding block procedures[49]. The confusion in this regard can be avoided to a great extent by developing a framework for direct estimation of crest settlement.

Swaisgood[53] studied the factors that influence crest settlement using 70 incidents of performance of embankments or dams in past earthquakes and proposed a relationship between estimated crest deformation and the peak horizontal ground acceleration. In essence, the approach is based on a family of relationships between observed crest settlement and peak ground acceleration that depend on earthquake magnitude. Although crest settlements are expected to be smaller for embankment dams constructed with or underlain by stronger materials, the approach did not consider material strength as a parameter of significance. Possible influence of the relationship between the fundamental period of the structure and the predominant frequency of the earthquake on crest settlement was also not examined.

In this article an attempt has been made to identify the parameters that influence earthquake-related crest settlement, Δ and develop a simple procedure for estimating Δ. The study is based on observational records on performance of embankment dams during past earthquakes and the proposed procedure for estimating crest settlement approximately accounts for the strength of materials within and underneath the dam body as well as the intensity of the earthquake. The case histories used in this study pertain to earthquakes of magnitudes between 4.5 and 8.3, peak horizontal ground accelerations between 0.02 and 0.90 g and a wide variety of earth structures such as single zone earth embankments, multi-zone earth and rock fill dams for retaining water, Concrete Face Rock Fill Dams (CFRDs) and tailings dams (Table 1).

Table 1: Case history listing

Dam, type, height (m)	Earthquake: Date, M_w, a_{max} (g), Dist. (km), T_p (s)	T_D (s), a_y (w/o and with vert. acc^n) (g)	Δ (m)	Reference
Anderson, 8, 73.2	10/17/89, 7.0, 0.26, 16, 0.32	1.08, 0.34, 0.24	0.0410	Harder[16]
Anderson, 8, 71.6	4/24/84, 6.2, 0.41, 16, 0.32	1.08, 0.27, 0.20	0.0140	Bureau *et al*.[6]
Artichoke, 2, 4.0	10/17/89, 7.1, 0.33, 27, 0.32	0.08, 0.28, 0.21	0.6000	Miller and Roycroft[31,32]
Austrian, 7, 21.5	10/17/89, 7.0, 0.58, 11, 0.32	0.79, 0.21, 0.17	0.7890	Harder[16]
Asagawara regulatory, 7, 56.4	10/23/04, 6.8, 0.12, 24, 0.32	0.53, 0.08, 0.07	0.7000	Yasuda *et al*.[55]
Baihe, 7, 66.0	7/28/76, 7.8, 0.20, 150, 0.52	0.89, 0.06, 0.06	2.5000	Lingyao *et al*.[25]
Bouquet Canyon, 5, 62.0	7/21/52, 7.3, 0.12, 74, 0.40	0.54, 0.16, 0.14	0.0010	Seed *et al*.[48]
Brea, 7, 27.4	1/17/94, 6.9, 0.19, 67, 0.45	0.76, 0.25, 0.17	0.0010	Abdel-Ghaffar and Scott[1]
Buena Vista, 5, 6.0	7/21/52, 7.3, 0.30, 32, 0.32	0.45, 0.16, 0.13	0.6000	Seed *et al*.[48]
Chabbot, 5, 43.3	4/18/06, 8.3, 0.57, 32, 0.32	0.99, 0.12, 0.11	0.4500	Makdisi and Seed[26]
Chabbot, 5, 43.3	10/17/89, 7.0, 0.10, 60, 0.32	0.99, 0.12, 0.11	0.0010	Makdisi and Seed[26]
Chang, 7, 15.5	1/26/01, 7.6, 0.50, 13, 0.32	0.25, 0.05, 0.05	2.6400	Singh *et al*.[50]
Chofukuji, 7, 27.2	10/23/04, 6.8, 0.10, 21, 0.32	0.38, 0.09, 0.08	0.0700	Yasuda *et al*.[55]
Chonan, 4, 6.1	12/17/87, 6.7, 0.12, 40, 0.32	0.11, 0.01, 0.01	3.8700	Ishihara *et al*.[20]
Cogoti D/S, 9, 85.0	4/4/43, 7.9, 0.19, 89, 0.60	0.83, 0.28, 0.23	0.3500	Arrau *et al*.[3]
Cogoti D/S, 9, 85.0	3/28/65, 7.1, 0.04, 153, 0.55	0.83, 0.28, 0.24	0.0010	Arrau *et al*.[3]
Cogoti D/S, 9, 85.0	7/8/75, 7.5, 0.05, 165, 0.57	0.83, 0.28, 0.24	0.0010	Arrau *et al*.[3]
Cogoti D/S, 9, 85.0	3/8/85, 7.7, 0.03, 280, 0.96	0.83, 0.28, 0.24	0.0010	Arrau *et al*.[3]
Cogswell, 9, 85.0	10/1/87, 6.0, 0.06, 29, 0.25	0.69, 0.13, 0.11	0.0010	Boulanger *et al*.[5]
Cogswell, 9, 8.50	6/28/91, 5.6, 0.26, 4, 0.25	0.69, 0.15, 0.14	0.0160	Boulanger *et al*.[5]
Demi 1, 7, 17.0	1/26/01, 7.6, 0.20, 90, 0.55	0.23, 0.26, 0.24	0.0500	Krinitzsky and Hynes[23]
Douhe, 4, 16.0	7/28/76, 7.8, 0.90, 20, 0.30	0.22, 0.34, 0.24	1.6400	Yen[60]
Dry Canyon, 5, 22.0	7/21/52, 7.3, 0.12, 72, 0.28	0.65, 0.12, 0.10	0.0300	Seed *et al*.[48]
El Cobre, 12, 32.5	3/28/65, 7.2, 0.80, 40, 0.32	0.49, 0.00, 0.00	32.000	Dobry and Alvarez[10]
El Infiernillo D/S, 8, 148.0	3/14/79, 7.6, 0.23, 110, 0.55	1.58, 0.55, 0.39	0.0460	Resendiz *et al*.[43]
El Infiernillo U/S, 8, 146.0	10/11/75, 5.9, 0.08, 79, 0.34	1.58, 0.08, 0.08	0.0400	Swaisgood[53]
El Infiernillo U/S, 8, 146.0	11/15/75, 7.5, 0.09, 23, 0.32	1.58, 0.09, 0.08	0.0200	Swaisgood[53]
El Infiernillo U/S, 8, 148.0	3/14/79, 7.6, 0.23, 110, 0.55	1.58, 0.19, 0.18	0.1280	Resendiz *et al*.[43]
El Infiernillo U/S, 8, 146.0	10/25/81, 7.3, 0.05, 81, 0.34	1.58, 0.05, 0.03	0.0600	Swaisgood[53]
El Infiernillo U/S, 8, 146.0	9/19/85, 8.1, 0.13, 76, 0.53	1.58, 0.11, 0.10	0.1100	Swaisgood[53]
El Infiernillo U/S, 8, 148.0	9/19/89, 8.1, 0.20, 113, 0.70	1.58, 0.13, 0.12	0.0490	Resendiz *et al*.[43]
El Infiernillo U/S, 8, 148.0	9/21/89, 7.2, 0.12, 116, 0.56	1.58, 0.13, 0.12	0.0650	Resendiz *et al*.[43]
El Khattabi, 10, 27.5	2/24/04, 6.4, 0.25, 21, 0.25	0.37, 0.18, 0.16	0.0100	EERI[14]
EJ Chesbro, 7, 29.0	4/24/84, 6.2, 0.18, 22, 0.32	0.37, 0.14, 0.13	0.0200	Swaisgood[53]

Table 1: Continue

EJ Chesbro, 7, 29.0	10/17/89, 7.0, 0.43, 13, 0.32	0.46, 0.15, 0.13	0.1130	Harder[16]
Fairmont, 5, 40.0	7/21/52, 7.3, 0.18, 36, 0.32	0.54, 0.11, 0.09	0.0010	Seed *et al*.[48]
Fatehgadh, 7, 11.6	1/26/01, 7.6, 0.30, 80, 0.55	0.28, 0.07, 0.07	1.0300	Singh *et al*.[50]
Gongen, 8, 32.6	1/17/95, 8.2, 0.11, 28, 0.32	0.40, 0.46, 0.32	0.0010	Matsumoto *et al*.[27]
Guadalupe, 7, 43.3	10/17/89, 7.0, 0.43, 19, 0.32	0.68, 0.19, 0.16	0.1950	Harder[16]
Guldurek, 7, 68.0	6/6/00, 5.9, 0.13, 19, 0.27	0.91, 0.27, 0.20	0.0200	Ozkan and Aksar[40]
Hachiro Gata, 4, 4.0	5/26/83, 7.7, 0.17, 95, 0.60	0.08, 0.00, 0.00	2.5400	Olson[38]
Hawkins, 6, 22.0	10/17/89, 7.0, 0.23, 34, 0.32	0.15, 0.30, 0.22	0.0010	Harder[16]
Hebgen, 7, 27.5	8/17/59, 7.5, 0.70, 100, 0.65	0.47, 0.22, 0.20	1.9200	Seed *et al*.[48]
Hokkaido tailings, 12, 9.2	5/16/68, 7.9, 0.23, 18, 0.60	0.14, 0.00, 0.00	12.500	Ishihara *et al*.[21]
Idenoshiri-Ike, 5, 5.5	1/17/95, 7.1, 0.44, 10, 0.32	0.65, 0.11, 0.09	4.0000	Uchida *et al*.[56]
Industrial, 2, 8.0	10/17/89, 7.1, 0.33, 18, 0.32	0.12, 0.35, 0.25	0.4000	Miller and Roycroft[31,32]
Ishibuchi, 9, 53.0	5/26/03, 7.1, 0.27, 85, 0.42	0.63, 0.28, 0.23	0.0010	Nagayama *et al*.[34]
Kalaghoda, 7, 14.9	1/26/01, 7.6, 0.30, 65, 0.47	0.18, 0.29, 0.24	0.0250	Krinitzsky and Hynes[23]
Kalpong, 9, 27.0	9/14/02, 6.5, 0.10, 21, 0.27	0.35, 0.10, 0.09	0.0010	Rai and Murty[42]
Kanayatani, 10, 4.0	10/6/00, 7.3, 0.11, 14, 0.32	0.05, 0.04, 0.04	0.7500	Matsuo[28]
Kashi, 7, 16.0	8/23/85, 7.4, 0.25, 21, 0.32	0.25, 0.14, 0.12	0.4000	Chonggang[8]
Kashi, 7, 16.0	9/12/85, 6.8, 0.50, 16, 0.30	0.25, 0.14, 0.12	1.5000	Chonggang[8]
Kaswati, 7, 12.	1/26/01, 7.6, 0.28, 110, 0.60	0.33, 0.15, 0.14	1.2100	Singh *et al*.[50]
Kawanishi, 7, 43.0	10/23/04, 6.8, 0.14, 17, 0.32	0.59, 0.14, 0.13	0.3000	Yasuda *et al*[55]
Kitayama, 7, 25.0	1/17/95, 7.1, 0.30, 31, 0.32	0.34, 0.15, 0.13	0.7500	Sakamoto *et al*.[44]
Kodanuma, 4, 2.5	5/16/68, 7.9, 0.23, –, 0.27	0.05, 0.00, 0.00	1.2900	Mishima and Kimura[30]
Kushiro Dike, 1, 6.4	1/15/93, 7.8, 0.20, 19, 0.32	0.11, 0.04, 0.06	2.0000	Sasaki *et al*.[46]
La Marquesa, 7, 10.0	3/3/85, 7.8, 0.67, 45, 0.40	0.11, 0.02, 0.02	2.0500	De Alba *et al*.[9]
La Palma, 7, 10.0	3/3/85, 7.8, 0.46, 80, 0.38	0.12, 0.05, 0.04	0.6100	De Alba *et al*.[9]
La Villita, 8, 60.0	11/15/75, 5.9, 0.08, 10, 0.25	0.94, 0.20, 0.15	0.0240	Elgamal *et al*.[11]
La Villita, 8, 60.0	10/11/75, 4.9, 0.15, 52, 0.27	0.94, 0.20, 0.17	0.0240	Elgamal *et al*.[11]
La Villita, 8, 60.0	3/14/79, 7.6, 0.100, 11, 0.55	0.94, 0.30, 0.19	0.0130	Elgamal *et al*.[11]
La Villita, 8, 60.0	10/25/81, 7.3, 0.17, 121, 0.60	0.94, 0.20, 0.18	0.1140	Elgamal *et al*.[11]
La Villita, 8, 60.0	9/19/85, 8.1, 0.24, 58, 0.48	0.94, 0.20, 0.17	0.3360	Elgamal *et al*.[11]
La Villita, 8, 60.0	9/21/85, 7.5, 0.04, 61, 0.32	0.94, 0.20, 0.18	0.1200	Swaisgood[53]
Lake Merced, 4, 12.5	3/22/57, 5.3, 0.12, 5, 0.32	0.27, 0.00, 0.00	6.6600	Olson[38]
Lake Temes, 3, 35.0	4/18/06, 7.6, 0.35, 2, 0.4	0.60, 0.10, 0.08	0.0100	Seed *et al*.[48]
Lexington, 7, 62.5	10/17/89, 7.0, 0.45, 10, 0.32	0.77, 0.13, 0.09	0.2590	Harder[16]
Long Valley, 7, 38.4	5/27/80, 6.1, 0.20, 16, 0.25	0.52, 0.23, 0.17	0.0010	Lai and Seed[24]
Lake Franklin, 5, 31.0	1/17/94, 6.7, 0.30, 18, 0.25	–, 0.28, 0.20	0.0500	Seed *et al*.[48]
Lower Van Norman, 7, 24.0	2/9/71, 6.6, 0.60, 13, 0.27	0.38, 0.19, 0.15	0.1440	Chaney[7]
Lower San Fernando, 6, 32.8	2/9/71, 6.6, 0.45, 8, 0.27	0.48, 0.00, 0.00	7.9500	Seed *et al*.[47]
Lower San Fernando, 6, 32.8	1/17/94, 6.9, 0.32, 11, 0.32	0.48, 0.15, 0.14	0.1500	Bardet and Davis[4]
LA dam, 7, 47.3	1/17/94, 6.9, 0.43, 7, 0.32	0.60, 0.15, 0.13	0.0880	Seed *et al*.[48]
Mahgoan, 3, 16.9	5/22/97, 6.0, 0.50, 38, 0.25	0.52, 0.29, 0.19	0.0200	EERI[12]
Makubetsu, 7, 26.9	9/26/03, 8.0, 0.25, 141, 0.50	0.42, 0.18, 0.17	0.5000	Nagayama *et al*.[34]
Matahina, 8, 86.0	3/2/87, 6.5, 0.24, 11, 0.28	1.08, 0.17, 0.14	0.0990	Pender and Robertson[41]
Matiyari, 3, 29.0	5/22/97, 6.0, 0.45, 95, 0.27	0.49, 0.27, 0.18	0.0100	EERI[12]
May 1 Slide, 11, 32.0	1/23/89, 5.5, 0.15, 3, 0.25	0.37, 0.00, 0.00	5.9200	Ishihara *et al*.[21]
Metoki, 4, 5.0	5/16/68, 7.9, 0.23, 180, 0.32	0.14, 0.00, 0.00	5.0000	Ishihara *et al*.[21]
Miboro, 8, 131.0	8/19/61, 7.0, 0.15, 16, 0.32	1.43, 0.23, 0.16	0.0260	Bureau *et al*.[6]
Miho, 8, 95.0	1/29/80, 6.6, 0.03, 57, 0.33	1.22, 0.23, 0.10	0.0010	Iwashita *et al*.[22]
Miho, 8, 95.0	4/14/81, 4.5, 0.03, 13, 0.32	1.22, 0.23, 0.17	0.0010	Iwashita *et al*.[22]
Miho, 8, 95.0	8/8/83, 6.0, 0.15, 12, 0.25	1.22, 0.23, 0.17	0.0010	Iwashita *et al*.[22]
Miho, 8, 95.0	12/17/87, 6.6, 0.01, 131, 0.6	1.22, 0.23, 0.20	0.0010	Iwashita *et al*.[22]
Miho, 8, 95.0	8/5/90, 5.1, 0.03, 24, 0.25	1.22, 0.23, 0.17	0.0010	Iwashita *et al*.[22]
Miho, 8, 95.0	2/2/92, 5.7, 0.01, 73, 0.32	1.22, 0.23, 0.19	0.0010	Iwashita *et al*.[22]
Mill Creek, 6, 23.2	10/17/89, 7.0, 0.28, 29, 0.32	0.30, 0.14, 0.12	0.0150	Harder[16]
Minase, 9, 67.0	6/16/64, 7.5, 0.08, 145, 0.77	0.72, 0.14, 0.10	0.0600	Swaisgood[53]
Minoogawa, 8, 47.0	1/17/95, 7.1, 0.14, 48, 0.34	0.57, 0.49, 0.40	0.0010	Matsumoto *et al*.[27]
Mochikoshi 1, 12, 30.0	1/14/78, 7.0, 0.25, 8, 0.32	0.42, 0.00, 0.00	22.750	Okusa and Anma[36]
Mochikoshi 2, 12, 22.0	1/14/78, 7.0, 0.25, 8, 0.32	0.42, 0.00, 0.00	15.900	Okusa and Anma[36]
Murayama, 7, 39.0	9/1/23, 8.2, 0.80, 96, 0.60	0.52, 0.29, 0.24	1.2000	Seed *et al*.[48]
Muraya-kami, 7, 24.0	9/1/23, 8.2, 0.80, 75, 0.60	0.52, 0.14, 0.11	0.1800	Stroitel[52]
Muraya-shino, 7, 30.0	9/1/23, 8.2, 0.80, 85, 0.60	0.52, 0.11, 0.10	0.0010	Stroitel[52]
Nalband, 4, 4.0	12/7/88, 6.8, 0.75, 28, 0.32	0.24, 0.00, 0.00	3.0000	Yegian *et al*.[58]
Newell, 8, 55.5	10/17/89, 7.0, 0.43, 10, 0.32	0.75, 0.25, 0.18	0.0110	Harder[16]
Niteko Loer, 3, 12.0	1/17/95, 6.9, 0.40, 4, 0.32	0.21, 0.07, 0.06	2.0000	Sitar[51]
Niteko middle, 3, 10.0	1/17/95, 6.9, 0.40, 4, 0.32	0.17, 0.04, 0.04	2.7000	Sitar[51]
Niteko upper, 3, 10.0	1/17/95, 6.9, 0.40, 4, 0.32	0.17, 0.04, 0.04	2.7000	Sitar[51]
Niwa Ikumine, 7, 15.0	7/12/93, 7.8, 0.28, 71, 0.36	0.23, 0.07, 0.04	1.7500	Tani[54]
North Dike, 36.0	1/17/94, 6.7, 0.43, 9, 0.32	0.45, 0.11, 0.09	0.0300	Swaisgood[53]

Table 1: Continue

O' Neil, 3, 21.3	10/17/89, 7.0, 0.11, 59, 0.33	0.37, 0.17, 0.14	0.0010	Harder[16]
Ono, 7, 36.6	9/1/23, 8.2, 0.80, 96, 0.60	0.52, 0.27, 0.24	0.3050	Seed *et al*.[48]
Oya, 8, 40.5	12/8/93, 5.0, 0.004, 42, 0.26	0.51, 0.20, 0.16	0.0010	Iwashita *et al*.[22]
Oya, 8, 40.5	2/16/93, 5.0, 0.01, 28, 0.25	0.51, 0.20, 0.16	0.0010	Iwashita *et al*.[22]
Oya, 8, 40.5	2/2/93, 4.8, 0.02, 9, 0.25	0.51, 0.20, 0.15	0.0010	Iwashita *et al*.[22]
Oya, 8, 40.5	2/7/93, 6.5, 0.07, 31, 0.28	0.51, 0.20, 0.16	0.0010	Iwashita *et al*.[22]
Oya, 8, 40.5	2/8/93, 4.9, 0.007, 37, 0.25	0.51, 0.20, 0.16	0.0010	Iwashita *et al*.[22]
Oya, 8, 40.5	6/7/94, 4.9, 0.005, 40, 0.25	0.51, 0.20, 0.16	0.0010	Iwashita *et al*.[22]
Oroville, 7, 235.0	8/1/75, 6.0, 0.11, 7, 0.25	2.74, 0.21, 0.13	0.0070	Bureau *et al*.[6]
Otani-Ike, 7, 27.0	12/21/46, 8.3, 0.80, 45, 0.32	0.65, 0.15, 0.12	0.0010	Stroitel[52]
Piedmont, 3, 17.0	4/18/06, 7.6, 0.35, 18, 0.40	0.60, 0.14, 0.13	0.2000	Seed *et al*.[48]
Route 272, 4, 7.5	1/15/93, 7.8, 0.38, 20, 0.40	0.13, 0.00, 0.00	5.2500	Miura *et al*.[33]
Rudramata, 7, 27.6	1/26/01, 7.6, 0.30, 80, 0.55	0.28, 0.07, 0.07	0.8300	Singh *et al*.[50]
San Andreas, 3, 32.0	4/18/06, 7.6, 0.80, 2, 0.40	0.60, 0.11, 0.09	0.0010	Seed *et al*.[48]
San Justo, 8, 41.0	10/17/89, 7.0, 0.26, 27, 0.32	0.51, 0.27, 0.21	0.0010	Harder[16]
San Luis, 3, 93.0	10/17/89, 7.0, 0.06, 54, 0.33	1.32, 0.09, 0.09	0.0010	Harder[16]
Santa Flacia, 6, 65.0	1/17/94, 6.7, 0.18, 33, 0.25	0.82, 0.17, 0.12	0.0200	Swaisgood[53]
Santa Flacia, 6, 72.0	9/2/71, 6.6, 0.11, 10, 0.25	0.78, 0.10, 0.08	0.0200	Abdel-Ghaffar and Scott[1]
Santa Flacia, 6, 72.0	4/8/76, 4.6, 0.05, 14, 0.25	0.78, 0.05, 0.04	0.0100	Abdel-Ghaffar and Scott[1]
Sasoi, 7, 20.0	1/26/01, 7.6, 0.20, 120, 0.63	0.27, 0.30, 0.28	0.0250	Krinitzsky and Hynes[23]
Shibecha Cho, 4, 9.5	1/15/93, 7.8, 0.38, 40, 0.40	0.16, 0.00, 0.00	9.2600	Miura *et al*.[33]
Shin-Yamam., 8, 44.5	10/23/04, 6.8, 0.55, 6, 0.32	0.56, 0.36, 0.24	0.0200	Yasuda *et al*.[55]
Shiribeshi Toshibetsu Dike 1, 1, 6.5	7/12/93, 7.8, 0.18, 100, 0.60	0.09, 0.04, 0.04	2.7000	Ozutsumi *et al*.[40]
Shiribeshi Toshibetsu Dike 2, 1, 4.5	7/12/93, 7.8, 0.18, 100, 0.60	0.07, 0.08, 0.08	1.2600	Ozutsumi *et al*.[40]
Shiribeshi Toshibetsu Dike 3, 1, 4.6	7/12/93, 7.8, 0.18, 100, 0.60	0.07, 0.12, 0.09	0.6300	Ozutsumi *et al*.[40]
Shivlakha, 7, 18.0	1/26/01, 7.6, 0.45, 28, 0.32	0.26, 0.23, 0.21	1.6200	Singh *et al*.[50]
Soda Lake, 13, 10.7	10/17/89, 7.0, 0.33, 29, 0.32	0.19, 0.16, 0.15	0.6000	Miller and Roycroft[31, 32]
Solfatara, 1, 5.0	5/18/40, 7.1, 0.33, 19, 0.32	0.05, 0.01, 0.01	2.0000	Olson[38]
South Haiwee, 5, 25.0	7/21/52, 7.3, 0.08, 151, 0.54	0.64, 0.11, 0.08	0.0200	Swaisgood[53]
South Levee, 2, 8.0	10/17/89, 7.1, 0.33, 18, 0.32	0.12, 0.29, 0.22	0.5000	Miller and Roycroft[31, 32]
Sugatadani-Ike, 12.0	1/17/95, 7.1, 0.23, 24, 0.28	0.54, 0.20, 0.18	2.0000	Uchida *et al*.[56]
Surajbari, 4, 8.0	1/26/01, 7.6, 0.35, 40, 0.32	0.12, 0.10, 0.09	0.3000	EERI[13]
Surgu, 8, 55.0	5/5/86, 6.6, 0.21, 10, 0.32	0.72, 0.15, 0.14	0.1500	Ozkan *et al*.[39]
Suvi, 7, 15.0	1/26/01, 7.6, 0.42, 37, 0.32	0.24, 0.09, 0.08	1.1000	Singh *et al*.[50]
Takami, 8, 120.0	9/26/03, 8.0, 0.33, 140, 0.50	1.31, 0.37, 0.31	0.0010	Nagayama *et al*.[34]
Tapar, 7, 15.5	1/26/01, 7.6, 0.15, 43, 0.32	0.21, 0.12, 0.07	0.8000	Singh *et al*.[50]
Tejon storage, 5, 11.0	7/21/52, 7.3, 0.60, 9, 0.28	0.45, 0.19, 0.16	0.0100	Seed *et al*.[48]
Tokachi Dike, 1, 6.0	9/27/03, 8.1, 0.40, 125, 0.70	0.09, 0.06, 0.06	2.0000	UJNRPW&SE[57]
Tokiwa, 7, 33.5	1/17/95, 7.1, 0.20, 10, 0.32	0.49, 0.20, 0.16	0.0010	Matsumoto *et al*.[27]
Torish Dike 1, 1, 5.2	1/17/95, 6.9, 0.22, 40, 0.32	0.15, 0.05, 0.04	3.0000	Ozutsumi *et al*.[41]
Torishima Dike 2, 1, 5.5	1/17/95, 6.9, 0.22, 40, 0.32	0.14, 0.07, 0.07	0.3000	Ozutsumi *et al*.[41]
Tsuboyama, 7, 20.5	10/23/04, 6.8, 0.13, 19, 0.32	0.29, 0.13, 0.12	0.0700	Yasuda *et al*.[55]
Upper crystal springs, 3, 25.0	4/18/06, 7.6, 0.80, 2, 0.40	0.60, 0.08, 0.07	0.0010	Seed *et al*.[48]
Upper Howell, 3, 13.0	4/18/06, 7.6, 0.80, 2, 0.40	0.60, 0.18, 0.15	1.6000	Seed *et al*.[48]
Upper San Fernando, 6, 25.0	2/9/71, 6.6, 0.45, 11, 0.32	0.38, 0.13, 0.11	0.9000	Seed *et al*.[47]
Upper San Fernando, 6, 25.0	1/17/94, 6.7, 0.32, 11, 0.27	0.46, 0.11, 0.11	0.1500	Bardet and Davis[4]
Vasona, 7, 10.4	10/17/89, 7.0, 0.40, 9, 0.32	0.19, 0.46, 0.31	0.0500	Harder[16]
Waste water plant, 2, 4.5	10/17/89, 7.1, 0.33, 23, 0.32	0.08, 0.50, 0.45	0.0200	Miller and Roycroft[31,32]
Yamam regulatory, 7, 27.2	10/23/04, 6.8, 0.55, 7, 0.32	0.38, 0.10, 0.09	0.5000	Yasuda *et al*.[55]
Yumig., 4, 7.5	10/6/00, 7.3, 0.30, 20, 0.32	0.13, 0.07, 0.06	1.0000	Matsuo[28]

(1): Dam types: 1: 1-zone levee, 2: Multi zone levee, 3: 1-zone earth dam, 4: 1-zone embankment, 5: 1-zone hydraulic fill dam; 6: Multi zone hydraulic fill; 7: Compacted multi zone dam; 8: Multi zone rock fill dam; 9 Concrete Faced Rock fill Dam (CFRD); 10: Concrete faced decomposed granite or gravel dam; 11: Natural slope; 12: Upstream constructed tailings dam; 13: Downstream constructed tailings dam; (2): $a_{max:}$ values in italics indicate cases where ground motions were measured at dam site; (3): Displacements in italics indicate cases involving liquefaction

MATERIALS AND METHODS

Intuitively, crest settlement is expected to be influenced by the intensity of the earthquake (measured by peak ground acceleration), the strength of the materials within and underneath the embankment dam, the relationship between the fundamental period of the embankment dam and predominant period of the earthquake and the earthquake magnitude (quasi resonance). The methods used in this study to estimate or quantify these parameters and assess their influence on earthquake-related crest settlements are summarized in the following subsections.

Peak ground acceleration: The peak ground acceleration, a_{max}, has been used in this study as a measure of the earthquake load intensity. The estimates for a_{max} were available from the references cited in Table 1. As indicated in Table 1, in only a few cases the earthquake accelerations were obtained from recorders located in close vicinity of the embankment or dam. Typically earthquake accelerations were estimated from attenuation relationships specifically developed for the earthquake considered in the reference and rarely from acceleration records from instruments installed near the dams or embankments.

Yield acceleration: Yield acceleration has been used in this study as a convenient, single-valued index of the shear strength of the material within and underneath the dam body. The parameter has been estimated following the sliding block approach as outlined below.

The stability assessment of the 152 case histories is based on the Newmark[35] sliding block approach. This conceptual framework approximates the potential sliding mass as a rigid body resting on a rigid sloping base. The contact between the potential sliding mass and the underlying slope is assumed as rigid-plastic. In this approach, the potential sliding mass is considered to mobilize irreversibly in the down slope direction when ground acceleration in that direction exceeds the threshold required to overcome the cohesive-frictional resistance at the base of the sliding mass. For a single pulse of down slope earthquake acceleration, the instantaneous velocity of the sliding mass relative to the sloping base is obtained by integration of the difference between the earthquake acceleration and 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 resistance and eventually may lose mobility. To obtain the magnitude of incremental, relative, down slope displacement of the sliding mass for the earthquake acceleration pulse that mobilized it, 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 earthquake.

The threshold acceleration above which the sliding mass is mobilized down slope, called the yield acceleration, is usually estimated from pseudo static slope stability analysis. The inertial effect due to the earthquake is typically accounted for by including the horizontal seismic coefficient, which when multiplied by the weight of the potential sliding mass (the volume of soil above the trial sliding surface and below slope face) provides a crude approximation of the average inertial force. Yield acceleration is taken to be equal to the horizontal seismic coefficient for which the factor of safety against slope instability is unity. Although the influence of the vertical component of earthquake-related ground motion is often neglected, this factor can be approximately accounted for by including a vertical seismic coefficient.

The pseudo static slope stability analysis is similar to static limit equilibrium slope stability assessment procedures except that the pseudo static procedure considers the inertial load during an earthquake on the potential siding mass usually by applying a single-valued horizontal acceleration sometimes referred to as the horizontal seismic coefficient. As in a static limit equilibrium analysis, the factor of safety is calculated against slope instability. The pseudo static analysis in this study was done with the help of slope stability software XSTABL[19]. The Simplified Bishop method was adopted in these computations. The soil properties used as inputs in these analyses were obtained as discussed in the following subsection.

In the first series of analyses, only the horizontal seismic coefficients were used. In the second series of analyses the vertical seismic coefficients were also included. Except for the few instances where the vertical as well as horizontal accelerograms were available at the site of the dam or embankment, the vertical seismic coefficients were assumed to be in proportion of the corresponding horizontal seismic coefficient depending on the distance of dam from the epicenter. The ratio between vertical and horizontal seismic coefficients was assumed to be 0.83 for a site to earthquake source distance of up to 10 km and a ratio of 0.33 was assumed for the distance of 100 km. For intermediate distances the ratio was assumed based on linear interpolation between the values mentioned above.

Soil properties: In terms of the shear strength and unit weights, the database assembled in this study is reasonably well constrained. In approximately 75% of the case histories, the shear strength and unit weights of soil or rock fill within and underneath the embankment or dam body were estimated from site and material specific Standard Penetration Test (SPT) blow counts, $(N_1)_{60}$, Cone Penetration Test (CPT) tip resistance,q_{c1}. For the remaining cases material specific test data were not directly available. To analyze these dams, generic properties were used. Such uncertainties in the input soil parameters exist in many projects at the preliminary design stage as in Sigh *et al*.[50].

For soils underneath water table, the pre- and post-liquefaction shear strengths were estimated following Olson and Stark[37] for soils characterized with normalized SPT blow count, $(N_1)_{60}$, of up to 12 or normalized cone tip resistance, q_{c1}, of up to 6.5 MPa. Soils with greater penetration resistance are considered non liquefiable. For non liquefiable soils, the shear strength was estimated using McGregor and Duncan[29] where penetration data were available unless the parameters were directly available from the case history reference. Range of shear strength and other input parameters associated with different types of dam, embankment or foundation materials are listed in the Table 1.

Dam fundamental (elastic) period: The fundamental (elastic) period, T_D, of the embankment-foundation system is estimated using the framework proposed by Gazetas and Dakoulas[15] for isotropic linear elastic embankments within a valley. For embankment dams, rockfill dams and CFRDs, valley geometry between narrow and wide trapeziums was assumed. For highway and railway embankments, river dikes and tailings dams the embankment was assumed to be infinitely long. The analytical expression for the fundamental period of an infinitely long dam or embankment, $T_{d\infty}$, measured in second is given by:

$$T_{d\infty} = 2.61H / V_s \quad (1)$$

Where:
H = The height of the dam or embankment in meter
V_s = The shear wave velocity in meter per second

The chart used for this purpose is shown in Fig. 1 for embankments or dams with crest lengths of up to ten times their maximum heights. In essence, the relationship shown in Fig. 1 represents valley geometry between narrow and wide trapeziums. Dams or embankments with crest length exceeding ten times the maximum height were considered to be infinitely long.

Shear wave velocity required as an input in Eq. 1 is estimated using the following empirical relationship[2]:

$$V_s = 93.2 \times (N_1)_{60}^{0.231} \quad (2)$$

where, $(N_1)_{60}$ is the normalized SPT blow count representative of the dam body materials.

Earthquake predominant period: In most of the cases, the predominant period of the earthquake ground motion, T_p, were estimated as per the Idriss[18] guidelines. In a few instances, predominant period of earthquake ground motion, T_p, was reported in the references based on acceleration records from strong motion instruments at dam sites.

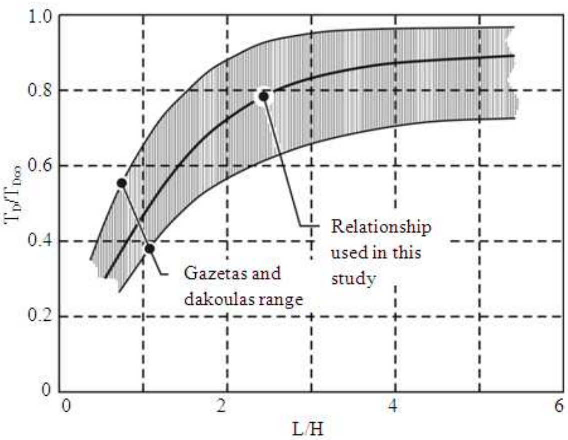

Fig. 1: Estimation of fundamental period

Parameters: If the parameters can be combined to obtain dimensionless hybrids based on simple physical considerations, assessment of parametric influence on crest settlement is expected to become convenient. Two such hybrid parameters are introduced in the following paragraphs. The extent of influence of two additional parameters, earthquake magnitude and the vertical component of ground motion, on crest settlement is also examined.

The crest settlement is expected to relate negatively to the shear strength of material within and underneath the embankment dam and positively to the ground motion amplitude. To assess the joint influence of the shear strength of material within and underneath the embankment dam and the ground motion amplitude on crest settlement the ratio of yield acceleration and peak horizontal acceleration, a_y/a_{max}, has been used following Hynes-Griffin and Franklin[17].

Secondly, in the events when the fundamental period of the embankment or dam is approximately equal to the predominant period of the earthquake, the impact of the earthquake is expected to be magnified because of quasi resonance. It therefore appears that crest settlements may depend on the ratio, T_D/T_p. The influence of this ratio on the crest settlement has also been studied.

RESULTS

The analytical results and observed crest settlements are listed in Table 1. Simple interpretation of these results is presented in the following subsections.

Material shear strength and horizontal component of ground motion: The observed crest settlements from Table 1, Δ, are plotted a_y/a_{max} in Fig. 2.

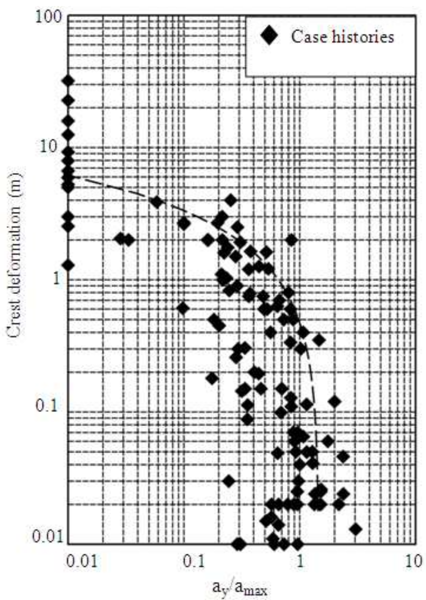

Fig. 2: Δ-a_y/a_{max} relationship without considering vertical ground acceleration

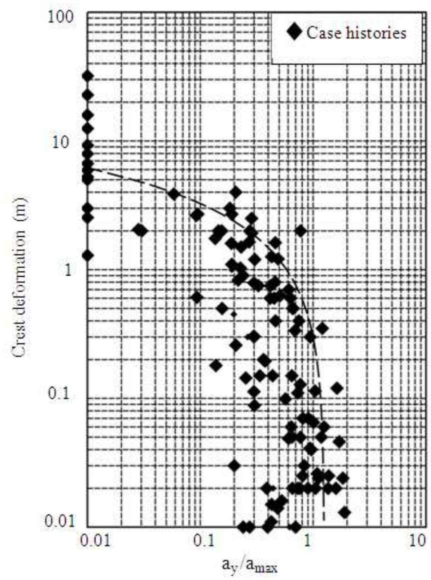

Fig. 3: Δ-a_y/a_{max} relationship considering vertical ground acceleration

Although there is a considerable scatter in the data, there appears to be a strong negative relationship between Δ and a_y/a_{max}. These results also provide a useful practical guidance by demonstrating that the likelihood of large crest settlements becomes small if the yield acceleration exceeds about 1.3 times the peak horizontal ground acceleration. Figure 2 also shows that the crest settlement sharply decreases as a_y/a_{max} exceeds 0.6 and becomes smaller than 0.2 as a_y/a_{max} exceeds unity. Also, the crest settlement could exceed 1 m if a_{max} was to exceed about 1.3 times the yield acceleration. These observations are consistent with the intuition that as the material within and underneath the embankment becomes strong relative to the intensity of earthquake load, the crest settlement tends to decrease.

It should be noted that the analyses upon which these inferences are based are physically simplistic. Also, there is a considerable scatter in the results shown in Fig. 2 with the correlation shown on the plot roughly representing the upper bound values of crest settlements. Thus, the correlation of Fig. 2 should not be viewed as a precise estimator of crest settlement; it, in essence, is an inexpensive tool that gives a reasonably conservative estimate of the crest settlements for an embankment dams under earthquake loading. The possibility of explaining the scatter by the differences in the peak vertical ground acceleration (i.e., whether the event is near field or far field), the earthquake magnitude (i.e., duration of earthquake) and the proximity of the fundamental period of the embankment dam and the predominant period of the earthquake is examined below.

Vertical component of ground motion, earthquake magnitude, predominant period of earthquake and fundamental period of embankment dam: To examine whether peak vertical ground acceleration explains the scatter, the results of analyses which included the peak vertical ground acceleration are shown in Fig. 3. A comparison of these results with Fig. 2 shows that inclusion of vertical ground acceleration does not improve the reliability in the prediction of crest settlement based on a_y/a_{max}. It appears therefore that peak vertical ground acceleration does not explain the scatter in the a_y/a_{max}-Δ data.

The variation of Δ with magnitude, M_w, Fig. 4, shows although there is a minor increasing trend Δ with increasing M_w, the influence of M_w on Δ may not be significant. Therefore, M_w does not appear to explain the scatter in the a_y/a_{max}-Δ data.

The variation of Δ against T_D/T_p, Fig. 5, shows that Δ decreases significantly as the ratio, T_D/T_p, becomes larger than 2. For smaller values of T_D/T_p the ratio does not appear to have a significant influence on Δ. Therefore variation in M_w also appears to account for the scatter in the a_y/a_{max}-Δ data partially.

Suggested use of the results: The relationship between a_y/a_{max} and Δ shown in Fig. 2 is based on simplified assumptions regarding material behavior implicit in the pseudostatic and sliding block frameworks. These assumptions are by themselves likely to be major contributors to the scatter in the results shown in Fig. 2.

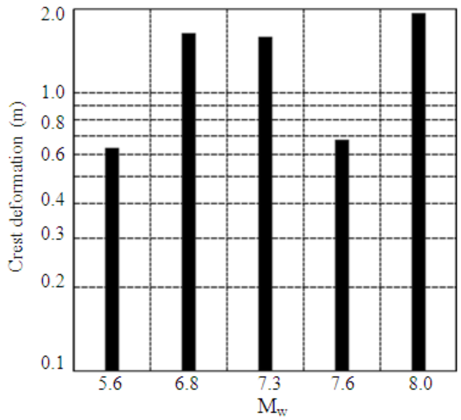

Fig. 4: Influence of M_w on crest settlement

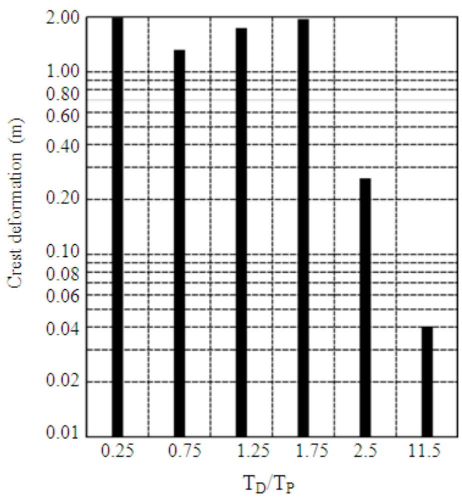

Fig. 5: Influence of T_D/T_P on crest settlement

Approximation of the earthquake load intensity with peak ground acceleration and the imprecision in the relationships of Eq. 1 and 2 used to estimate the fundamental period of embankment dam and shear wave velocity, respectively, are also expected to contribute to the uncertainty in the a_y/a_{max}-Δ correlation.

Since it is difficult to quantify these uncertainties, a more sophisticated statistical treatment of the results was not considered necessary. Further analysis of the results using a soft computation tool, e.g., the artificial neural network or the fuzzy logic framework, is also unlikely to enhance the practical usefulness of the procedure proposed in this article because of the conceptual simplifications of the analytical model used in this study.

Although the uncertainty in the proposed a_y/a_{max} -Δ relationship limits its potential as a detailed engineering tool, the relationship is expected to be useful in several practical problems. The a_y/a_{max}-Δ correlation, for instance, could be used as an inexpensive tool for assessing retrofit requirement of an existing embankment dam or for quick estimation of freeboard requirement of an embankment dam in the early planning stage.

DISCUSSION

The objective of this article was to identify the main parameters that influence earthquake-related crest settlements of embankment dams and first-order quantification of their influence on crest settlement. Observed performance of earth dams and embankments in 152 instances of earthquake loading was considered for the purpose.

The results of the study indicate that the ratio of yield acceleration and peak horizontal ground acceleration, a_y/a_{max}, has a strong influence on earthquake-related crest settlement, Δ. An approximate correlation between a_y/a_{max} and Δ has been derived from these results indicated that crest settlement sharply decreases as a_y/a_{max} exceeds 0.6 and becomes smaller than 0.2 as a_y/a_{max} exceeds 1.0. The crest settlement could exceed 1 m if a_{max} was to exceed about 1.3 times the yield acceleration. Although there is a considerable scatter in the a_y/a_{max}-Δ data, which prevents the use of the relationship as a detailed engineering tool, it is believed that the relationship could be used as an easy and inexpensive screening tool in the preliminary design stage of a proposed embankment dam or in resource planning for a dam retrofit project. The conceptual simplicity of the analytical procedure used in this study is likely to be a major contributor to the scatter in the a_y/a_{max}-Δ data. Since the uncertainty in the a_y/a_{max}-Δ relationship on this count is not easily quantifiable, the data were not subjected to a rigorous and elaborate treatment for developing a more precise estimator of earthquake-related crest settlement.

The influence of T_D/T_p, M_w and the vertical component of ground motion on crest settlements were also assessed. These assessments indicated that the crest settlements are insensitive to T_D/T_P for $T_D/T_p \leq 2$ but there is a sharp decrease in crest settlement as T_D exceeds $2T_p$. The influence of M_w was found to be less remarkable and only a small increase in the crest settlement was apparent with increasing earthquake magnitude. The influence of vertical acceleration on crest settlement also appeared to be minor. Consequently, there appears to be no added benefit in including the vertical component of ground motion in pseudo static slope stability analysis.

To summarize, a detailed evaluation of earthquake induced crest settlement is only appears to be needed if (a) a_y/a_{max} exceeds 1 or (b) $T_D \leq 2T_p$.

CONCLUSION

Performance records of embankments and dams from past earthquakes indicate that there is a strong correlation between a_y/a_{max} and crest settlement. Crest settlement also appears to depend on T_D/T_P and marginally on earthquake magnitude.

REFERENCES

1. Abdel-Ghaffar, A.M. and R.F. Scott, 1979. Analysis of earth dam response to earthquakes. J. Geotech. Eng., 105: 1379-1404. http://cedb.asce.org/cgi/WWWdisplay.cgi?5015033
2. Andrus, R.D. and K.H. Stokoe, 2000. Liquefaction resistance of soils from shear-wave velocity. J. Geotech. Geoenviorn. Eng., 126: 1015-1025. DOI: 10.1061/(ASCE)1090-0241(2000)126:11(1015)
3. Arrau, L., Ibarra, I. and G. Noguera, 1985. Performance of Cogoti Dam Under seismic Loading. In: Concrete Face Rockfill Dams-Design, Construction and Performance, Barry Cooke, J. and J.L. Sherard (Eds.). ASCE., ISBN: 0872625036, pp: 1-14.
4. Bardet, J.P. and C.A. Davis, 1996. Performance of San Fernando dams during 1994 Northridge earthquake. J. Geotech. Geoenviorn. Eng., 122: 554-563. DOI: 10.1061/(ASCE)0733-9410(1996)122:7(554)
5. Boulanger, R.W., J.D. Bray, S.M. Merry and L.H. Mejia, 1995. Three-dimensional dynamic response analysis of Cogswell dam. Can. Geotech. J., 32: 452-464. DOI: 10.1139/t95-048
6. Bureau, G., R.L. Volpe, W.H. Roth and T. Udaka, 1985. Seismic Analysis of Concrete face Rockfill Dams. In: Concrete Face Rockfill Dams-Design, Construction and Performance, Barry Cooke, J. and J.L. Sherard (Eds.). ASCE., ISBN: 0872625036, pp: 479-508.
7. Chaney, R.C., 1979. Earthquake induced deformation in earth dams. Proceeding of the 2nd US National Conference on Earthquake Engineering, pp: 633-642.
8. Choggang, S., 1988. Some experiences from damages of embankments during strong earthquakes in China. Proceeding of the 2nd International Conference on Case Histories in Geotech. Earthquake Engineering, pp: 729-735.
9. De Alba, P.A., H.B. Seed, E. Retamal and R.B. Seed, 1988. Analyses of dam failures in 1985 Chilean earthquake. J. Geotech. Eng., 114: 1414-1434. DOI: 10.1061/(ASCE)0733-9410(1988)114:12(1414)
10. Dobry, R. and L. Alvarez, 1967. Seismic failures of Chilean tailings dams. J. Soil Mech. Found. Div., 93: 237-260.
11. Elgamal, A.W., R.F. Scott and M.F. Succarieh, 1990. La Villita dam response during five earthquakes including permanent deformation. J. Geotech. Eng., 114: 1443-1462. DOI: 10.1061/(ASCE)0733-9410(1990)116:10(1443)
12. Earthquake Engineering Research Institute, 1997. Some observation on engineering aspects of the Jabalpur earthquake of 22 May, 1997. http://nicee.org/eqe-iitk/uploads/EQR_Jabalpur.pdf
13. Earthquake Engineering Research Institute, 2001. The Republic Day (Bhuj) earthquake of Kachchh and active faults, Gujarat, western India. Reconnaissance report. EERI, Oakland, CA, USA. http://sciencelinks.jp/j-east/article/200123/000020012301A0929620.php
14. Earthquake Engineering Research Institute, 2004. Preliminary observation on the Al Hoceima Morocco earthquake of 24 February, 2004. Special report. EERI, Oakland, CA, USA. http://www.eeri.org/lfe/pdf/morocco_alhoceima_eeri_preliminary_report.pdf
15. Gazetas, P. and P. Dakoulas, 1992. Seismic analysis and design of rockfill dams. Soil Dyn. Earthquake Eng., 11: 27-61. http://cat.inist.fr/?aModele=afficheN&cpsidt=4731618
16. Harder Jr., L.H., 1991. Performance of earth dams during Loma Prieta earthquake. US Geological Survey Professional Paper, 1552: 3-26. http://cat.inist.fr/?aModele=afficheN&cpsidt=9984042
17. Hynes-Griffin, M.E. and A.G. Franklin, 1984. Rationalizing the seismic coefficient method. Miscellaneous Paper: GL-84-13, US Army Corps of Engineers Waterways Experiment Station, Vicksburg, Mississippi. http://oai.dtic.mil/oai/oai?verb=getRecord&metadataPrefix=html&identifier=ADA144730
18. Idriss, I.M., 1991. Earthquake ground motions at soft soil sites. Proceeding of the 2nd International Conference on Recent Advances in Geotech. Earthquake Engineering and Soil Dynamics, Mar. 11-15, St. Louis, Missouri, pp: 2265-2272. http://nisee.berkeley.edu/elibrary/Text/LIB050207
19. Interactive Software Designs, Inc., 1994. XSTABL: An integrated slope stability analysis program for personal computers. Reference Manual. Interactive Software Designs, Inc., Moscow, Idaho, USA.

20. Ishihara, K., S. Yasuda and Y. Yoshida, 1990. Liquefaction-induced flow failure of embankments and residual strength of silty sands. Soils Found., 30: 69-80. http://ci.nii.ac.jp/naid/110003959719/ja
21. Ishihara, K., S. Okusa, N. Oyagi and A. Ischuk, 1990. Liquefaction-induced flow slide in the collapsible loess deposit in Soviet Tajik. Soils Found., 30: 73-89. http://ci.nii.ac.jp/naid/110003959974/en
22. Iwashita, T., N. Yasuda, A. Nakamura and O. Takeda, 1995. Dynamic analysis of rockfill dams considering frequency dependent characteristics of radiation damping ratio. Proceeding of the 1st International Conference on Earthquake Geotech. Engineering, pp: 511-516.
23. Krinitzsky, E.L. and M.E. Hynes, 2002. The Bhuj, India, earthquake: Lessons learned for earthquake safety of dams on alluvium. Eng. Geol., 66: 163-196. DOI: 10.1016/S0013-7952(02)00049-2
24. Lai, S.S. and H.B. Seed, 1980. Dynamic response of long valley dam in the Mammoth lake earthquake series of May 25-27 1980. Report 85/12, Univ. of Calif., Berkeley. http://nisee.berkeley.edu/documents/EERC/EERC-85-12.pdf
25. Lingyao, L., L. Kueifen and B. Dongping, 1980. Earthquake damage of Baihe earth dam and liquefaction characteristics of sand and gravel materials. Proceeding of the 7th World Conference on Earthquake Engineering, Istanbul, pp: 171-178.
26. Makdisi, F.I. and H.B. Seed, 1978. Simplified procedure for estimating dam and embankment earthquake-induced deformations. J. Geotech. Eng., 104: 849-867. http://cedb.asce.org/cgi/WWWdisplay.cgi?5013898
27. Matsumoto, M., A. Nakamura, T. Sasaki and T. Iwashita, 1996. Effects on Dams. Soil Found., 36: 273-281. http://ci.nii.ac.jp/naid/10016595629
28. Matsuo, O., 2000. Report on the Western Tottori Prefecture earthquake of October 6, 2000. In: Report of 33rd Tech. of US-Japan Natural Resources Development Program, Japan. http://www.jstage.jst.go.jp/article/seisankenkyu/52/12/52_590/_article
29. McGregor, J.A. and J.M. Duncan, 1998. Performance and use of standard penetration tests I geotechnical engineering practice. Report of Virginia Tech Center for Geotech. Practice and Research.
30. Mishima, S. and H. Kimura, 1970. Characteristics of landslides and embankment failures during the Tokachioki earthquake. Soils Found., 10: 39-51. http://ci.nii.ac.jp/naid/110003899025/
31. Miller, E.A. and G.A. Roycroft, 2004. Seismic Performance and Deformation of Leeves: Four case studies. J. Geotech. Geoenviorn. Eng., 130: 344-354. DOI: 10.1061/(ASCE)1090-0241(2004)130:4(344)
32. Miller, E.A. and G.A. Roycroft, 2004. Compaction grouting test program for liquefaction control. J. Geotech. Geoenviorn. Eng., 130: 355-361. DOI: 10.1061/(ASCE)1090-0241(2004)130:4(355)
33. Miura, K., N. Yoshida and K. Wakamatsu, 1995. "Damage to fill embankment during the 1993 Kushiro-oki earthquake. Proceeding of the 1st International Conference on Earthquake Geotech. Engineering, pp: 1057-1062. ISBN: 905410578X
34. Nagayama, I., Y. Yamaguchi, T. Sasaki, A. Nakamura, H. Kawasaki and D. Hirayama, 2004. Damage to dams due to three large earthquake occurred in 2003, in Japan. US-Japan Natural Resources Development Program, Japan. http://www.pwri.go.jp/eng/ujnr/joint/36/paper/63nagaya.pdf
35. Newmark, N.M., 1976. Effects of earthquakes on dams and embankments. Géotechnique, 15: 139-160. http://nisee.berkeley.edu/elibrary/Text/2006091413
36. Okusa, S. and S. Anma, 1980. Slope failures and tailings dam damage in the 1978 Izu-Ohshima-Kinkai earthquake. Eng. Geol., 16: 195-224. DOI: 10.1016/0013-7952(80)90016-2
37. Olson, S.M. and T.D. Stark, 2003. Yield strength ratio and liquefaction analyses of slopes and embankments. J. Geotech. Geoenviorn. Eng., 129: 727-737. DOI: 10.1061/(ASCE)1090-0241(2003)129:8(727)
38. Olson, S.M. 2001. Liquefaction analysis of level and sloping ground using field case histories and penetration resistance. PhD Dissertation, University of Illinois, Urbana-Champaign. https://netfiles.uiuc.edu/olsons/www/olson_thesis.pdf
39. Ozkan, M.Y., M. Erdik, M.A. Tuncer and C. Yilmaz, 1996. An evaluation of Surgu dam response during 5 May 1986 earthquake. Soil Dyn. Earthquake Eng., 15: 1-10. DOI: 10.1016/0267-7261(95)00030-5
40. Ozutsumi, O., S. Sawada, S. Iai, Y. Takeshima, W. Sugiyama and T. Shimazu, 2002. Effective stress analysis of liquefaction-induced deformation in river dikes. Soil Dyn. Earthquake Eng., 22: 1075-1082. DOI: 10.1016/S0267-7261(02)00133-1
41. Pender, M.J. and T.W. Robertson, 1987. Edgecumbe earthquake: Reconnaissance report. Bull. N. Z. Natl. Soc. Earthquake Eng., 20: 201-249.
42. Rai, D.C. and C.V.R. Murty, 2003. North Andaman (Diglipur) earthquake of 14 September 2002. India. http://nicee.org/eqe-iitk/uploads/EQR_Andaman.pdf

43. Resendiz, D., Romo, M.P. and E. Moreno, 1982. El infiernillo and la villita dams: Seismic behavior. J. Geotech. Eng., 108: 109-131. http://cedb.asce.org/cgi/WWWdisplay.cgi?8200034
44. Sakamoto, S., H. Yoshida, Y. Yamaguchi, H. Satoh, T. Iwashita and N. Matsumoto, 2002. Numerical simulation of sliding of an earth dam during the 1985 Kobe earthquake. Proceeding of the 3rd US-Japan Workshop on Advanced Research on Earthquake Engineering for Dams, June 22-23, San Diego, California, pp: 416. http://www.stormingmedia.us/68/6857/A685714.html
45. Sarma, S.K. 1975. Seismic stability of earth dams and embankments. Géotechnique, 25: 743-761.
46. Sasaki, Y., K. Tamura, M. Yamamoto and J. Ohbayashi, 1995. Soil improvement work for river embankment damaged by the 1993 Kushiro-Oki earthquake. Proceeding of the 1st International Conference on Earthquake Geotech. Engineering, pp: 43-48. http://river.ceri.geo.jp/itiran/h7/h7-189-202.pdf
47. Seed, H.B., K.L. Lee, I.M. Idriss and F.I. Makdisi, 1975. The slides in the San Fernando dams during the earthquake of February 9, 1971. J. Geotech. Geoenviorn. Eng., 101: 651-688. http://cedb.asce.org/cgi/WWWdisplay.cgi?5011449
48. Seed, H.B., I.F. Makdisi and P. De Alba, 1978. Performance of earthen dams during earthquakes. J. Geotech. Eng., 104: 967-994. http://cedb.asce.org/cgi/WWWdisplay.cgi?5013870
49. Singh, R., D. Roy and D. Das, 2007. A correlation for permanent earthquake-induced deformation of earth embankments. Eng. Geol., 90: 174-185. DOI:10.1016/J.ENGGEO.2007.01.002
50. Singh, R., D. Roy and S.K. Jain, 2005. Analysis of earth dams affected by the 2001 Bhuj earthquake. Eng. Geol., 80: 282-291. DOI:10.1016/J.ENGGEO.2005.06.002
51. Sitar, N., 1995. Geotechnical Reconnaissance of the effect of January 17, 1995 Hyogoken Nambu earthquake Japan. Report of EERC No. 95/01, University of Berkeley, USA. http://library.eerc.berkeley.edu/archives/kobe/EERC-95-01/
52. Stroitel, G., 1969. Design and investigation of seismically stable dams in Japan. Power Technol. Eng. (Formerly Hydrotech. Construct.), 3: 946-955. DOI: 10.1007/BF02395813
53. Swaisgood, J.R., 2003. Embankment dam deformations caused by earthquakes. Paper No. 14. http://db.nzsee.org.nz/PCEE/2003/View/Paper014s.pdf
54. Tani, S., 2000. Behavior of large fill dams during earthquake and earthquake damage. Soil Dyn. Earthquake Eng., 20: 223-229. DOI: 10.1016/S0267-7261(00)00055-5
55. Yasuda, N., M. Kondo, T. Sano, H. Yoshioka, Y. Yamaguchi, T. Sasaki and M. Tomita, 2005. Effect of the mid Niigata prefecture earthquake in 2004 on dams. US Japan Natural Resources Development Program, Technical Report. http://www.pwri.go.jp/eng/ujnr/joint/37/paper/a3yasuda.pdf
56. Uchida, K., T. Torri, T. Tsujino, S. Ando, K. Higasio and T. Yamamoto, 2001. A study of failure mechanism of embankment dam for irrigation damaged by the 1995 Hyogoken Nambu earthquake. Proceeding of the 4th Internatioanl Confernece on Recent Advances in Geotech. Earthquake Engineering.
57. UJNR Panel on Wind and Seismic Effects, 2003. Report on the 26 September, 2003 Tokachi-Oki Earthquake, Japan, http://www.pwri.go.jp/eng/ujnr/newnl/enl_4.pdf
58. Yegian, M.K., E.A. Marciano and V.G. Ghahraman, 1991. Earthquake-induced permanent deformations: Probabilistic approach. J. Geotech. Eng., 117: 35-50. http://cat.inist.fr/?aModele=afficheN&cpsidt=19533632
59. Yegian, M.K., V.G. Ghahraman and R.N. Harutinunyan, 1994. Liquefaction and embankment failure case histories, 1988 Armenia earthquake. J. Geotech. Eng., 120: 581-596. DOI: 10.1061/(ASCE)0733-9410(1994)120:3(581)
60. Yan, L., 1991. Seismic deformation analysis of earth dams: a simplified method. Report of Calif. Institute of Tech., Calif., Report SML 91-01. http://caltecheerl.library.caltech.edu/272/00/SML9101.pdf