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Integrity of Sub-Structural Systems during Earthquake: Indian and International Perspectives

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Abstract: Structural integrity of underground constructions or sub-structures like foundations, anchors, basements, piles, piers, abutments, retaining walls, subgrade of railway, highway, airport runway etc. are important concern for the engineers and researchers in the entire world to advance on the infrastructural developments of any country. Several historical giant earthquakes all over the world had always put the researchers and engineers to the new challenge for developing advanced and new techniques of design and construction to maintain the structural integrity of the infrastructural systems. In this paper, the state-of-the-art type analysis and design techniques for behavior of such sub-structural system during seismic events are discussed. In this connection, the role of the design codes in India with a comparison to such activities around the world using international design codes is discussed. The importance of soil profile with seismic characterization is highlighted. Benefits of the recently proposed and validated pseudo-dynamic approach over the conventional pseudo-static approach have been revealed. The mitigation technique such as the use of geosynthetic materials for the stability and integrity of such sub-structures during seismic events is also discussed. The latest trends in research and practice for design of sub-structural systems in India and other countries are revealed.

Keywords: Earthquake geotechnical engineering, design codes, pseudo-dynamic method, geosynthetics, earthquake resistant geotechnical structures.

1 Introduction

The devastating effect of earthquake is well known worldwide. Some of the few historical strong earthquakes (above Magnitude 8.0) which had made severe damages to the mankind are listed in Tab. 1. Also Tab. 2 shows the number of largest

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and deadliest earthquakes occurred worldwide during last ten years (2000 to 2010). By studying Tab. 2 carefully, one can easily understand that the largest earthquakes need not be always the deadliest earthquakes. Besides the number of population of the earthquake affected area, it is the damages of structures and sub-structures which are the reasons for the deadliest earthquakes even though the magnitudes may not be necessarily the largest in a particular year.

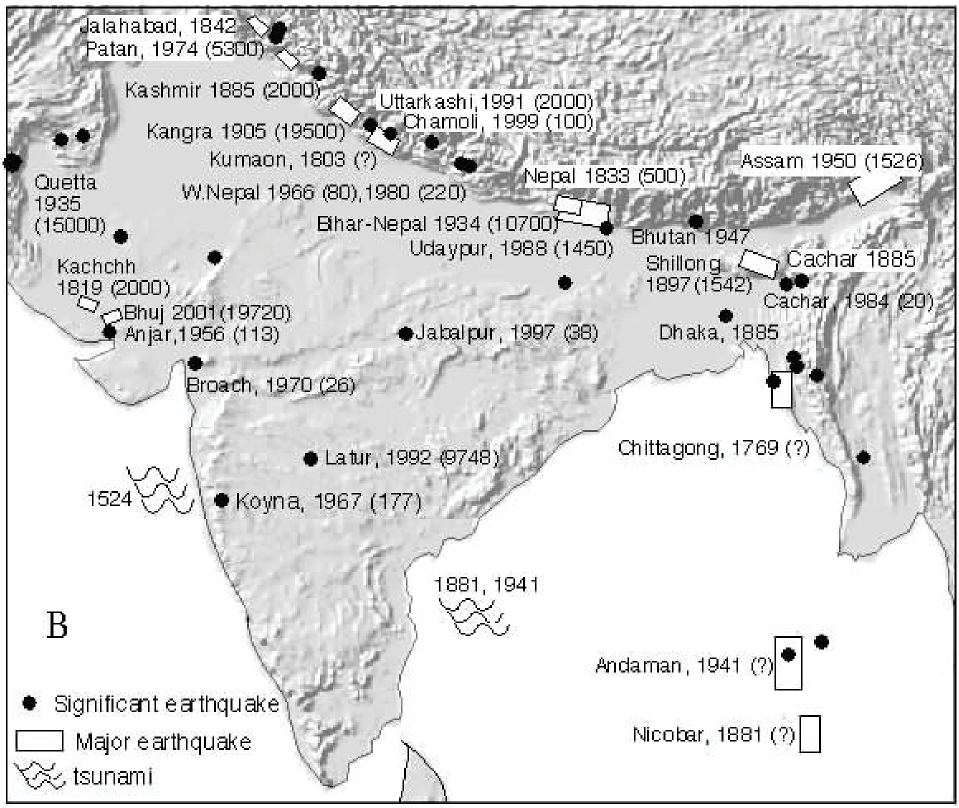


Figure 1: Location, year and number of fatalities (in parenthesis) for earthquakes in India during 1800 – 2001 (Modified after Bilham and Gaur, 2000)

Figure 1 shows the location, year and number of earthquake damages in India during years 1800 – 2001. It can be easily compared with the worldwide earthquake intensities and those occurred in India. It can be noticed that both large and devastating earthquakes are frequent and common in Indian scenario. This needs a retrospective in the earthquake resistant design of structures both above and below ground. It is known that the participation and experience of India in the world

Table 1: Historical strong earthquakes and damages worldwide (Source: US Geological Survey website, viz. http://earthquake.usgs.gov/)

Date	**Magnitude**	**Location**	**Effects**
October 20, 1687	8.5	Lima, Peru	Destroyed much of the city.
July 8, 1730	8.7	Valparasio, Chile	Killed about 3000 people.
November 1, 1755	8.7	Lisbon, Portugal	Also generated Tsunami and killed about 60,000 people and destroyed much of Lisbon.
November 7, 1837	8.5	Valdivia, Chile	Generated Tsunami and killed atleast 58 people in Hawaii.
August 13, 1868	9.0	Africa, Peru (currently in Chile)	Generated catastrophic Tsunami and kille about 25,000 people in South America.
June 15, 1896	8.5	Sanriku, Japan	Generated a Tsunami and killed atleast 22,000 people.
January 31, 1906	8.8	Off the coast of Ecuador and Colombia.	Generated Tsunami and killed atleast 500 people.
November 11, 1922	8.5	Chile-Argentina border	Killed several hundred people.
August 15, 1950	8.6	Assam, India and Tibet	Killed about 780 people.
May 22, 1960	9.5	Southern Chile	Also generated Tsunami and killed atleast 1,716 people.
March 27, 1964	9.2	Prince William Sound, Alaska	Also generated Tsunami and killed about 128 people.
December 26, 2004	9.0	Off the Indonesia Island of Sumatra	Triggered a Tsunami that killed about 226,000 people in 12 countries, including 165,700 in Indonesia and 35,400 in Sri Lanka.
February 27, 2010	8.8	Offshore Maule, Chile	Generated Tsunami and number of people killed and massive damages in Chile

Table 2: Worldwide largest and deadliest earthquakes during 2000 to 2010

Largest Earthquakes				Deadliest Earthquakes			
Date	**Magnitude**	**Fatalities**	**Region**	**Date**	**Magnitude**	**Fatalities**	**Region**
Feb. 27, 2010	8.8	507	Offshore Maule, Chile	January 12, 2010	7.0	222,570	Haiti
September 29, 2009	8.1	192	Samoa Islands region	September 30, 2009	7.5	1,117	Southern Sumatra, Indonesia
May 12, 2008	7.9	87,587	Eastern Sichuan, China	May 12, 2008	7.9	87,587	Eastern Sichuan, China
September 12, 2007	8.5	25	Southern Sumatera, Indonesia	August 15, 2007	8.0	514	Near the Coast of Central Peru
November 15, 2006	8.3	0	Kuril Islands	May 26, 2006	6.3	5,749	Java, Indonesia
March 28, 2005	8.6	1,313	Northern Sumatra, Indonesia	October 8, 2005	7.6	80,361	Pakistan
December 26, 2004	9.1	227,898	Off West Coast of Northern Sumatra	December 26, 2004	9.1	227,898	Off West Coast of Northern Sumatra
September 25, 2003	8.3	0	Hokkaido, Japan Region	December 26, 2003	6.6	31,000	Southeastern Iran
November 3, 2002	7.9	0	Central Alaska	March 25, 2002	6.1	1,000	Hindu Kush Region, Afghanistan
June 23, 2001	8.4	138	Near Coast of Peru	January 26, 2001	7.7	20,023	Bhuj, India
November 16, 2000	8.0	2	New Ireland Region, P.N.G.	June 4, 2000	7.9	103	Southern Sumatera, Indonesia

forum for earthquake engineering research has tremendously improved in recent days (Kaushik and Jain, 2009). It is pleasant to note that the progress in the area of structural earthquake engineering is already reached a professional level in terms of applications through the use of design codes, however, the same for the underground or geotechnical structures takes a backseat not only in India but worldwide till last century. But several developed nations like USA, Japan, Europe and many others have developed and updated the design codes related to earthquake resistant design aspects for geotechnical or substructures like foundations, retaining walls, slopes, earth dams etc. in recent past. But in India, till date there is hardly any development in the front of geotechnical earthquake engineering in the application part through design codes, though several recent research works have been demonstrated to be fruitful in mitigation of such earthquake damages due to geotechnical failures such as liquefaction or ill-performance of foundations due to the dynamic behaviour of soil. To the engineering and practice oriented community, it is somewhat easy to adopt a generalized earthquake design techniques for superstructures which cannot be adopted for underground or geotechnical structures due to non-uniform variation of soil properties from place to place. It can be noted that the design procedure adopted worldwide for superstructures can be more or less similar in a design problem as guided in the codes. But for geotechnical or underground structures the design aspects vary completely from not only country to country but also from place to place even within same locality, due to the variation of soil profile both in depth and lateral directions. For example, a tall multistoried building in USA or in India can follow the same design principles for superstructure but the major difference occurs due to the difference in soil properties in the two countries which makes geotechnical earthquake engineering as a problem which is location specific. Hence proper design guidelines are necessary for Indian design codes to address geotechnical problems related to earthquake engineering as has been done by USA, Japan, Europe and many other countries worldwide. In this paper a retrospective for geotechnical earthquake engineering design provisions given in codes have been highlighted with a difference from similar guidelines in other countries with the proposed recent research guidelines which can be adopted further for mitigation of earthquake disasters in India.

The classification of soil properties and variation are not only extremely gross but also highly non-technical in the design code IS 1893 – Part 1 (2002). It describes the soil classification in just three major categories viz. hard rock, stiff soil and soft soil with some information about the ranges of standard penetration test (SPT) blow count number (N). As in field test it is very common to identify the strength of cohesionless soil by using SPT “N” values, but it is highly questionable to use such “N” values for other cohesive soils. Because SPT test is suitable for only cohesionless soil but not for cohesive soil (Bowles, 1996). To avoid the limitation of a particular test specific result, the worldwide design codes have re-classified recently the different types of soil to be considered for design based on shear wave velocity and site period which truly indicate the nature of the soil subjected to any earthquake or dynamic motion. For example, Tab. 3 shows the guidelines for the soil classification provided in the common design codes used in USA viz. NEHRP (1997) for geotechnical problems. It can also be noticed that the major six classifications of soil with different sub-classifications covers all technically possible ranges of soil profile subjected to earthquake loading, which is missing in the present Indian design code. Based on experience and international standards adopted, the proposed soil classification for Indian soils can be used as suggested in Tab. 4. It can be noted that to cover and use common field test results used in India, like SPT for cohesionless soil and Cone Penetration Test (CPT) for cohesive soil, both the ranges of values for various soil categories have been proposed to cover all ranges of soil like cohesionless to cohesive soils along with the ranges for shear wave velocities and site periods to consider the effect of earthquake loading on soil.

For earthquake resistant design of geotechnical substructures, till date in Indian codes (IS 1893 – Part 3) and many other countries adopt the age old Mononobe-Okabe method (see Okabe 1926; Mononobe and Matsuo 1929; Kramer 2005), which is based on pseudo-static approach (see Terzaghi, 1950). Also several researchers like Newmark (1965), Arya and Gupta (1966), Seed (1966), Prakash and Saran (1966), Prakash and Basavanna (1969), Madhav and Kameswara Rao (1969), Seed and Whitman (1970), Sarma (1975), Richards and Elms (1979), Nadim and Whitman (1983), Davies et al. (1986), Ebeling and Morrison (1992), Kramer and Smith (1997), Rathje and Bray (1999), Wu and Finn (1999), Zeng and Steedman (2000), Kumar (2001), Choudhury and Subba Rao (2002), Loukidis et al. (2003), Wartman et al. (2003), Subba Rao and Choudhury (2005), Psarropoulos et al. (2005), Choudhury and Singh (2006), Nouri et al. (2008) and many others had used pseudo-static approach for the seismic design of geotechnical structures like retaining walls, slopes, earth dams etc. but the use of pseudo-static approach found to be very limited. As can be seen that the pseudo-static approach neither considers any dynamic phenomenon of earthquake loading like duration of motion, period or frequency of motion, shear and primary wave velocities through the soil media, soil amplification etc. Pseudo-static approach only considers a single magnitude of earthquake accelerations which is added to the inertia component for the analysis to design such geotechnical structures (Steedman and Zeng, 1990). Also mostly the force-based design method using limit equilibrium method is widely adopted. But in geotechnical structures subjected to earthquake loading experience excessive dis-

Table 3: Geotechnical site categories (Bray and Rodriguez-Marek, 1997)

Site	**Description**	**Site Period**	**Comments**
A	Hard Rock	$\leq$ 0.1s	Hard, strong, intact rock ($V_s \geq 1500$ m/s)
B	Rock	$\leq$ 0.2s	Most "unweathered" California rock cases ($V_s \geq 760$ m/s or < 6 m of soil)
C-1	Weathered/Soft Rock	$\leq$ 0.4s	Weathered zone > 6 m and < 30 m ($V_s \geq$ 360 m/s increasing to $\geq$ 700 m/s)
C-2	Shallow Stiff Soil	$\leq$ 0.5s	Soil depth > 6 m and < 30 m
C-3	Intermediate Depth Stiff Soil	$\leq$ 0.8s	Soil depth > 30 m and < 60 m
D-1	Deep Stiff Holocene Soil, either Sand or Clay	$\leq$ 1.4s	Soil depth > 60 m and < 210 m, Sand has low fines content (<15%) or non-plastic fines (PI<5). Clay has high fines content (>15%) and plastic fines (PI>5)
D-2	Deep Stiff Pleistocene Soil, Sand or Clay	$\leq$ 1.4s	Soil depth > 60 m and < 210 m. see D1 for sand and clay classification
D-3	Very Deep Stiff Soil	$\leq$ 2s	Soil depth > 210 m
E-1	Medium Depth Soft Clay	$\leq$ 0.7s	Thickness of soft clay layer 3 m to 12 m
E-2	Deep Soft Clay Layer	$\leq$ 1.4s	Thickness of soft clay layer > 12 m
F	Special, e.g. Potentially Liquefiable Sand or Peat	$\approx$ 1s	Holocene loose sand with high water table (<6 m) or organic peats

placements in most of the damages. Hence a displacement-based design approach will be better than a force-based design approach for such geotechnical structures (Choudhury et al, 2004). In entire Europe, the design code, viz. Eurocode 8 (1998) also provides the guidelines similar to the use of displacement-based approach over force-based design for geotechnical structures under earthquake conditions.

To overcome the limitations of pseudo-static approach, recently Choudhury and Nimbalkar (2005, 2006, 2007, 2008) had proposed a complete pseudo-dynamic approach which considers not only the magnitude of earthquake accelerations but also the duration of motion, period or frequency of motion, shear and primary waves traveling through the soil media, soil amplification etc. in the analysis to result closed-form design solutions. In Europe, in design code viz. Eurocode 8

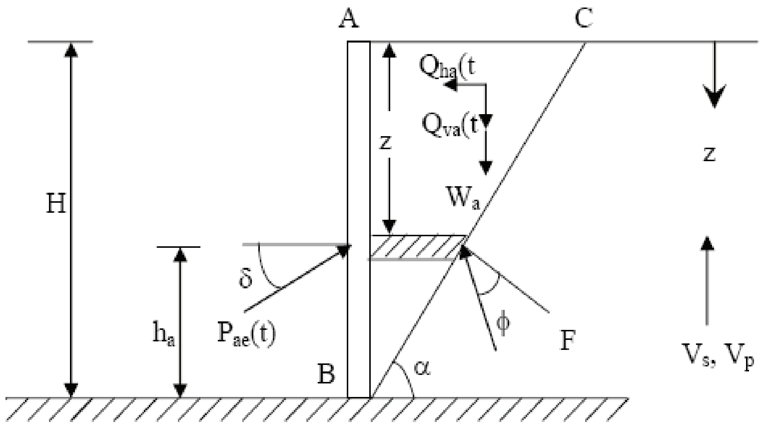
Table 4: Proposed Soil Classification for incorporation in IS 1893 (Part 1)

Site Classification	**Soil Types**	**N Value**	**Cone bearing q_c (MPa)**	**Shear wave velocity, V_s (m/sec)**	**Site Period (sec)**
A	Hard Rock	-	-	> 1500	< 0.1
B	Medium Rock	-	-	700 – 1500	< 0.2
C-1	Weathered/Soft Rock (weathered zone > 5 m)	> 40	-	350 – 700	< 0.4
C-2	Shallow Stiff Soil (soil depth 10m – 30m) (SP, SP with gravels with no fines)	> 30	> 4.0	300 – 350	< 0.5
C-3	Intermediate Depth Stiff Soil (soil depth 30m – 60m) (SP, SM with little fines)	20 – 30	0.9 – 4.0	250 – 300	< 0.8
D-1	Deep Stiff Soil (Sand or Clay) (soil depth 60m – 200m) (sand with low fines < 15%, non plastic PI < 5%) (clay with high fines > 15% and PI > 5%) (SM or ML or CL)	10 – 20		200 – 250	< 1.4
D-2	Very Deep Stiff Soil (Sand or Clay) (soil depth > 200m) (SC or ML or CL)			150 – 200	< 2.0
E-1	Medium Depth Soft Clay or Silt (soil depth 3m – 12m) (MI, MH, MI-CI, MH-CH)	-	< 0.9	< 150	< 0.7
E-2	Deep Soft Clay (soil depth > 12m) (CI, CH)	-			< 1.4
F	Potentially liquefiable sand or peat (high water table > 5m) (OI, OH)	< 10			About 1.0

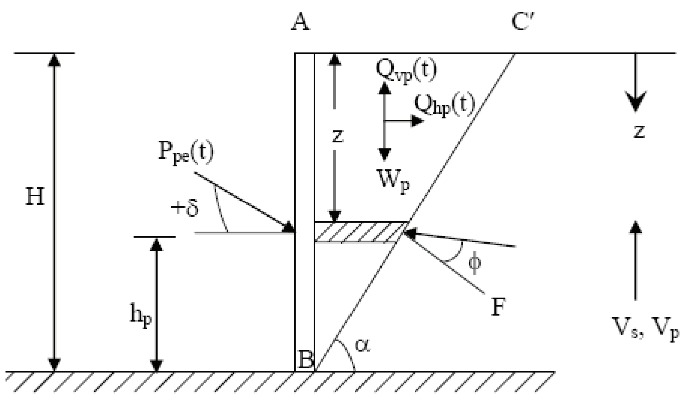
(1998) also proposes to use the soil amplification factor for the design of geotechnical structures under earthquake loadings. The use and effectiveness of this newly proposed pseudo-dynamic approach has been also experimentally verified using the available dynamic geotechnical centrifuge test results on model geotechnical structures in literature (Nimbalkar and Choudhury, 2008).

2 Pseudo-dynamic approach

To detail the newly developed pseudo-dynamic approach as originally proposed by Steedman and Zeng (1990) with further modification and consideration of both horizontal and vertical seismic accelerations and all body waves traveling during the earthquake motion and soil amplification is reported by Choudhury and Nimbalkar (2005, 2006, 2007, 2008) as shown in Figs. 2(a) and 2(b) for a rigid vertical cantilever retaining wall AB of height H, supporting cohesionless backfill. Similar to the pseudo-dynamic approach propsoed by Steedman and Zeng (1990) with horizontal seismic acceleration and shear wave velocity only, Choudhury and Nimbalkar (2005, 2006) had considered both finite shear wave velocity (V_s) and primary wave velocity (V_p) in the modified and latest improved pseudo-dynamic analysis. The phase and the magnitude of both the horizontal and vertical seismic accelerations are varying along the depth of the wall. In the present analysis, by using the relationship between the primary and shear wave velocities with Poisson's ratio (ν_s) of the material, V_p/V_s = 1.87 for ν_s = 0.3 is used. Period of lateral shaking, $T = 2\pi/\omega = 4H/V_s$ is considered in the analysis. The base of the wall is subjected to harmonic horizontal seismic acceleration of amplitude $k_h g$, where g is the acceleration due to gravity and harmonic vertical seismic acceleration of amplitude $k_v g$. The exact nature of soil amplification is dependent on many factors, including the geometry and rigidity of adjacent structures, the stiffness and damping in the soil, the depth of the soil layer and so on. Again a simplified assumption is made that the horizontal and vertical acceleration vary linearly from the input acceleration at the base to the higher value (depending upon the soil amplification) at the top of the retaining wall, such that $k_{h(z=0)} = f k_{h(z=H)}$ *and* $k_{v(z=0)} = f k_{v(z=H)}$, where f is a constant and is termed as amplification factor (Nimbalkar and Choudhury, 2008). For no amplification, f = 1.0. For obtaining the critical design value of seismic earth pressure, in the present analysis, it is assumed that both the horizontal and vertical vibrations with amplitude of accelerations $k_h g$ and $k_v g$ respectively, start at exactly the same time and there is no phase shift between these two vibrations to provide worst possible design combinations or critical condition. Referring to Figs. 2(a) and 2(b), the acceleration at any depth z and time t, below the top of the wall



(a)



(b)

Figure 2: (a) Model rigid retaining wall for computation of pseudo-dynamic active earth pressure (Choudhury and Nimbalkar, 2006). (b) Model rigid retaining wall for computation of pseudo-dynamic passive earth resistance (Choudhury and Nimbalkar, 2005)

can be expressed as,

$$a_h(z,t) = \left\{1 + \frac{H-z}{H}(f-1)\right\} k_h g \sin\omega\left(t - \frac{H-z}{V_s}\right) \tag{1}$$

$$a_v(z,t) = \left\{1 + \frac{H-z}{H}(f-1)\right\} k_v g \sin\omega\left(t - \frac{H-z}{V_p}\right) \tag{2}$$

Now, the total seismic active earth pressure (P_{ae}) (in Fig. 2a) on the retaining wall calculated by using the pseudo-dynamic approach is given by,

$$P_{ae} = \frac{1}{2} K_{ae} H^2 \bar{\gamma}(1 - r_u) \tag{3}$$

where $\bar{\gamma}$= modified unit weight of the backfill soil and it is actually the weighted average of the total unit weight of the backfill soil below and above the water table. $\bar{\gamma}$ is calculated using the approach suggested by Kramer (2005) as

$$\bar{\gamma} = \left(\frac{h_{wd}}{H}\right)^2 \gamma_{sat} + \left(1 - \left(\frac{h_{wd}}{H}\right)^2\right)\gamma_d \tag{4}$$

in which γ_d = dry unit weight of the backfill soil, γ_{sat} = saturated unit weight of the backfill soil, r_u = pore pressure ratio, K_{ae} = seismic active earth pressure coefficient calculated using the pseudo-dynamic approach (Choudhury and Nimbalkar, 2006) and given by

$$\left.\begin{array}{l} K_{ae} = \frac{1}{\tan\alpha_{ae}} \frac{\sin(\alpha_{ae}-\phi)}{\cos(\delta+\phi-\alpha_{ae})} + \\ \frac{k_h}{2\pi^2 \tan\alpha_{ae}} \left(\frac{TV_s}{H}\right) \frac{\cos(\alpha_{ae}-\phi)}{\cos(\delta+\phi-\alpha_{ae})} m_1 + \\ \frac{k_v}{2\pi^2 \tan\alpha_{ae}} \left(\frac{TV_p}{H}\right) \frac{\cos(\alpha_{ae}-\phi)}{\cos(\delta+\phi-\alpha_{ae})} m_2 \end{array}\right\} \tag{5}$$

with ϕ and δ = soil and wall friction angles, α_{ae} = wedge angle with the horizontal for the active case, then m_1 and m_2 appearing in Eq. (5), are given by

$$\begin{array}{l} m_1 = 2\pi\cos 2\pi\left(\frac{t}{T} - \frac{H}{TV_s}\right) + \\ \left(\frac{TV_s}{H}\right)\left(\sin 2\pi\left(\frac{t}{T} - \frac{H}{TV_s}\right) - \sin 2\pi\left(\frac{t}{T}\right)\right) \end{array} \tag{6}$$

$$\begin{array}{l} m_2 = 2\pi\cos 2\pi\left(\frac{t}{T} - \frac{H}{TV_p}\right) + \\ \left(\frac{TV_p}{H}\right)\left(\sin 2\pi\left(\frac{t}{T} - \frac{H}{TV_p}\right) - \sin 2\pi\left(\frac{t}{T}\right)\right) \end{array} \tag{7}$$

The simplicity and effectiveness of the pseudo-dynamic approach in the analysis and design of geotechnical structures like retaining wall, foundation, slope, anchors, waterfront retaining structures, landfills, earth dams etc. have been shown extensively in recent research findings worldwide (Choudhury and Nimbalkar 2007, Ghosh 2007, Choudhury and Ahmed 2007, 2008, Ahmed and Choudhury 2008, 2009, Basha and Babu 2009, Kolathayar and Ghosh 2009 and many others). This new design technology can be adopted for practice through design codes in India and other countries worldwide.

3 Mitigation

As a measure of mitigation of earthquake damages due to failure of geotechnical retaining structures, few researchers like Tatsuoka et al. (2007) and others had shown the better performance of reinforced soil-wall over the conventional retaining wall during 1995 Kobe earthquake in Japan. Bergado (2007) had shown the better performance of geosynthetic confinement systems during tsunami generated due to earthquake. Similar methodologies must be adopted worldwide by adopting pseudo-dynamic approach for the analysis and design of reinforced soil-wall (Nimbalkar et al. 2006, Choudhury et al. 2007, Ahmad and Choudhury 2008, Choudhury and Ahmad 2009, Reddy et al. 2009). It will additionally help to mitigate the disaster due to geotechnical damages.

4 Conclusions

In this paper a brief review of the design and practices of geotechnical earthquake engineering in India with comparison to various worldwide practices are reported. The role of soil characterization for design and construction of geotechnical structures in earthquake prone regions have been revealed. Use of shear wave velocity and site period based soil characterization seems to be better than using only static field test data. The advantages and benefits of more realistic dynamic analysis using pseudo-dynamic approach over conventional pseudo-static approach for geotechnical structures are mentioned. The mitigation of earthquake damages for geotechnical structures by using geosynthetics is mentioned. The most important aspects for the need of urgent revision and due consideration of geotechnical earthquake engineering in Indian design codes for earthquake resistant design and construction are highlighted.

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