International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

1995 - Third International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

04 Apr 1995, 10:30 am - 12:00 pm

On Seismic Design Displacements of Rigid Retaining Walls

Shamsher Prakash
Missouri University of Science and Technology, prakash@mst.edu

Yingwei Wu
University of Missouri--Rolla

Eyjolfur Ami Rafnsson
Honnum, Reykjavik, Iceland

Proceedings: Third International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics
April 2-7, 1995; Volume III, St. Louis, Missouri

On Seismic Design Displacements of Rigid Retaining Walls

Paper No. 4.19

Shamsher Prakash and Yingwei Wu
University of Missouri-Rolla
Rolla, MO, USA

Eyjolfur Arni Rafnsson
Honnum
Reykjavik, Iceland

ABSTRACT Correlation of the dynamic displacement and significant factors are presented. Three computer programs (Rafnsson, 1991) have been modified to develop design charts. The wall dimension computed by static condition and related displacement under dynamic loading can be estimated from the computer programs. Twenty-one combinations of base soil and back fill, 5 different ground motions and 7 different heights of wall are used in the analyses to develop design charts. These will help the designer to predict the dynamic behavior of retaining walls and to optimize the design work. Furthermore, equations have been fitted to predict the displacement without using the computer program in several cases.

INTRODUCTION

The conventional design of rigid retaining walls requires estimating the earth pressure behind a wall and choosing the wall geometry to satisfy specified factors of safety under static conditions. The factors of safety are used to prevent sliding, overturning and bearing capacity failure. The two classical earth pressure theories of Coulomb and Rankine have been applied for computation of the pressure. These theories provide safe estimates of the earth pressures on retaining structures under static conditions. However, the factors of safety will be decreased due to dynamic load.

During an earthquake, sudden increases in earth pressure develop. The magnitude of these dynamic increases was first studied by Okabe (1926) and later by Mononobe and Matsuo (1929). Their analytical method, known as the Mononobe-Okabe method, is widely used for dynamic lateral pressure computation and is a straightforward extension of the Coulomb sliding wedge theory where pseudo-static horizontal and vertical inertial forces of the fill material are included (Prakash 1981 a,b). The earth pressure can be defined as an active condition when the earthquake acceleration is toward the back fill and the wall moves away from it.

Amana, Azuma et al. (1956) discussed damage to gravity type quay-walls that took place during several earthquakes in Japan from 1930 to 1956. The walls moved laterally, tilted and settled in some cases. Shakya(1987) presented a compilation of damage to retaining walls that failed by sliding at the base and by combined sliding and overturning (Table 1).

The design method for a retaining wall still uses static or pseudo-static pressure to compute wall dimensions and determine the factors of safety. The displacement of the retaining structures under dynamic conditions have not been taken into account. However, dynamic loading is generated during earthquakes for a number of significant loading cycles. The static or pseudo-static method cannot assure that the displacement remains within acceptable limits during an earthquake. Therefore, knowledge of the displacement under dynamic conditions is becoming increasingly important for the design of retaining walls. Similar procedures will be used for design of abutments of bridges subjected to an earthquake.

Nandakumaran (1973) and Richards and Elms (1979) developed design procedures to account for displacements in sliding only. Nadim and Whitman (1984) developed a model for computed sliding and tilting. Rafnsson (1991) also incorporated both sliding and rotation in his analysis and developed a computer program. The factors affecting the displacement were studied in his work. However, both analyses were not in usable formats by the practicing engineer.

A computer program had been developed by Rafnsson (1991) to predict horizontal movement at the top of a retaining wall under dynamic loading due to simultaneous sliding and rocking motion. The factors affecting the displacement had been studied in his work.

The study of dynamic behavior of retaining walls using a computer program for different conditions will help practicing engineers to predict their behavior more precisely. Moreover, the data generated can be used for safer and more economical design of the retaining walls.

Rafnsson's work has been advanced in this investigation to include following :

(1) A comprehensive set of soil properties to represent field conditions of back fill and base soil was selected.

(2) Design tables are developed for displacement at the top of the retaining walls of different dimensions and loading conditions (i.e., magnitude M and ground motion α_h).

Rafnsson's computer programs were modified to generate appropriate design data and preparation

Table 1. Failures and movements of retaining structures during earthquakes (Shakya, 1987)

Earthquake	M.	Year	Place	Wall Type	Acc'n	Found. Soil	Reported Damage
Kitaizu	7.1	1930	Shimizu Harbor	Gravity			Walls moved 26 ft
Shizuoka		1935	Shimisu				Retaining wall collapse, moved 16 ft
Tonankai		1944	Yokkaic-hi Port	Trestle type Quaywalls		Soft clay underlain by alternate layers of gravel and clay	Feature of damage would imply the occurrence of sliding
Nankai	8.1	1946	Uno Port	Gravity	0.1g-0.5g	Refilled sand layer 26 ft thick overlain layer of clay to sandy-clay	Wall slid 35 cm toward the sea, little settlement
Tokachi-oki	7.8	1952	Kushiro Port	Concrete cassions, placed on a rubble mound		sand	Settlement sliding, and tilting outward.
Chile	8.4	1960	Puerto Montt	Gravity			Complete overturning
Niigata		1964	Central Pier in Niigata Port	Concrete block type	0.23g		Swelling of face line and settlement of fill, sliding and tilting toward the sea
Niigata	7.5	1964	Rinko district Niigata Port	Trestle type Quaywall	0.23g	Loose sand, N-value <5	Large settlement
Tokachi-oki	7.5	1968	Noheji Port	Concrete block Quaywalls	0.23g	Ground consists of very loose sandy alluvial layer	Swelling of face line and settlements of blocks
Tokachi-oki		1968	Hachinohe Port		0.26g		The walls titled 5° and swelled towards the sea 60 cm
Nemurohanto-oki		1973	Hanasaki Port	Concrete block Quaywall	0.28g		Swelling of face line was 60 cm and settlement of block of 30 cm
Miyagiken-oki	7.5	1978	Ishinomaki Port	Steel sheetpile			Slid toward the sea 57 to 119 cm

of design charts. This work will aid in safer and quicker design by the field engineer. Equations to predict the displacement were also developed.

RAFNSSON'S (1991) WORK

It is appropriate to describe Rafnsson's (1991) work to prepare the reader to understand the present investigation. The model consists of a rigid wall resting on the surface of the soil and subjected to horizontal exciting ground motion. The soil behavior is non-linear for both back fill and base soil. Both material and geometrical damping in sliding and rocking motions have been considered (Figure 1) (Rafnsson 1991, Rafnsson and Prakash 1994).

In Figure 1, K represents the stiffness of a material and c the damping of the soil. The mathematical model represents both the active case and the passive case (Figures 2,3). Nonlinear behavior of soil is included in defining the following properties, both at the base as well as the backfill:

(1) Soil stiffness in sliding.
(2) Soil stiffness in rocking.
(3) Geometrical damping in sliding.
(4) Geometrical damping in rocking.
(5) Material damping in sliding.
(6) Material damping in rocking.

The nH shown in Figures 2 and 3 is the height to any point on the back of the wall measured from the base. The n value was taken as 0.5 in the analysis model.

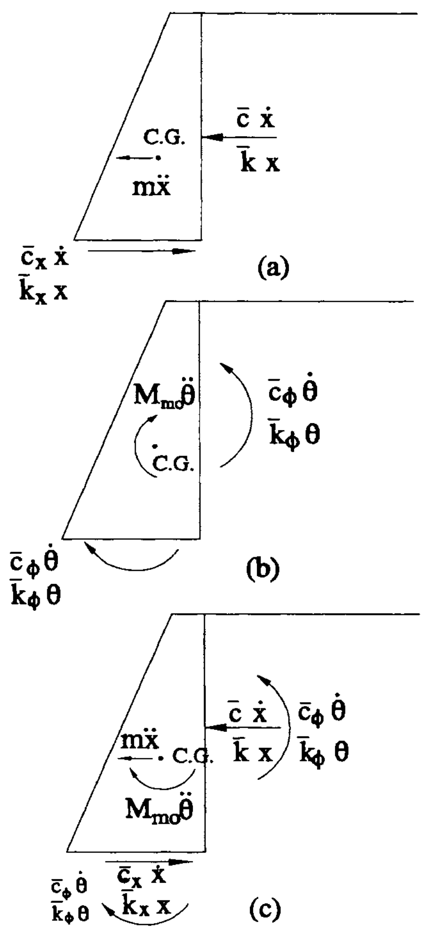


Figure 1. Mathematical model of retaining wall: (a)sliding only, (b)rocking only, (c)combined sliding and rocking (Rafnsson, 1991).

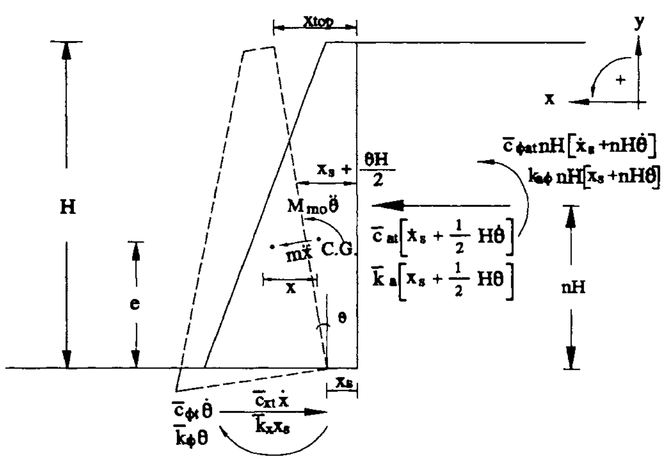


Figure 2. Mathematical model for stiffness and damping constants for the active case (Rafnsson 1991).

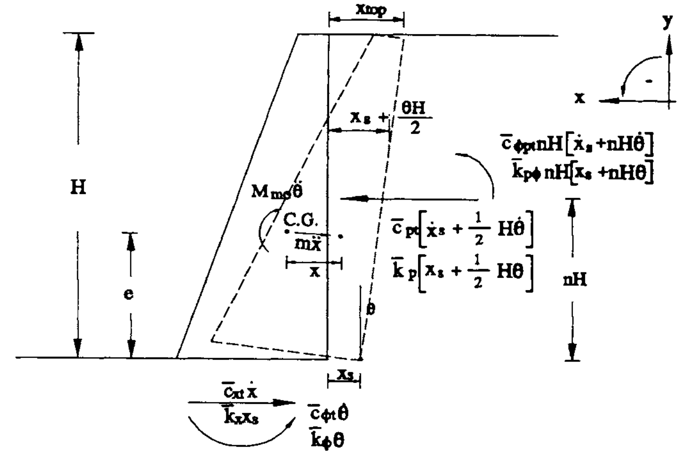


Figure 3. Mathematical model for stiffness and damping constants for the passive case (Rafnsson 1991).

The equations of motion for both horizontal sliding and rotation were written as:

For the active case:

$$M_{mo}\ddot{\theta} + c_R\dot{\theta} + k_R\theta - c_{HR}\dot{x}_s - k_{HR}x_s = M_x(t) \tag{1}$$

$$m\ddot{x}_s + c_x\dot{x}_s + k_x x_s + me\ddot{\theta} - c_{HS}\dot{\theta} - k_{HS}\theta = P_x(t) \tag{2}$$

For the passive case:

$$M_{mo}\ddot{\theta} + c_R\dot{\theta} + k_R\theta + c_{HR}\dot{x}_s + k_{HR}x_s = M_x(t) \tag{3}$$

$$m\ddot{x}_s + c_x\dot{x}_s + k_x x_s + me\ddot{\theta} + c_{HS}\dot{\theta} + k_{HS}\theta = P_x(t) \tag{4}$$

In the above equations, m represents the mass of the wall, M_{mo} - the mass moment of inertia, x_s the horizontal displacement, θ the angular rotation, and c the dynamic damping. Subscripts "HS" and "HR" represent total damping for backfill in sliding and rocking respectively, subscript "x" sliding, and "R" rotation. The stiffness (k) and damping (ξ) in several modes are both strain and frequency dependent and have been presented elsewhere (Rafnsson 1991 and Rafnsson and Prakash 1991).

For prediction of dynamic displacement of a rigid retaining wall, three computer programs were developed (Rafnsson, 1991) as follow
(1) input program,
(2) main program,
(3) check program.

Input Program (WALLINP.FOR).

1. This program will compute the base width and generate an input file for the main and check programs.

Table 2. Backfill and base soil combination, and dimension of retaining wall (Rafnsson, 1991)

case No	soil	$\phi°$	$\delta°$	e	γ (pcf*)	ν	Note
1	Back Base	33 33	22 22	0.65 0.65	100.2 100.2	0.313 0.313	Reference Wall
2	Back Base	30 33	20 22	0.75 0.65	94.5 100.2	0.333 0.313	Study the effect of various backfills and base soil(33°)
3	Back Base	28.5 33	19 22	0.85 0.65	89.3 100.2	0.343 0.313	
4	Back Base	36 33	24 22	0.58 0.65	104.7 100.2	0.292 0.313	
5	Back Base	33 30	22 20	0.65 0.75	100.2 94.5	0.313 0.333	Study the effect of various backfills and base soil (30°)
6	Back Base	30 30	20 20	0.75 0.75	94.5 94.5	0.333 0.333	
7	Back Base	28.5 30	19 20	0.85 0.75	89.3 94.5	0.343 0.333	
8	Back Base	36 30	24 20	0.58 0.75	104.7 94.5	0.292 0.333	
9	Back Base	36 28.5	24 19	0.58 0.85	104.7 89.3	0.292 0.343	Study the effect of various base soils
10	Back Base	36 36	24 24	0.58 0.58	104.7 104.7	0.292 0.292	
11	Back Base	28.5 28.5	19 19	0.85 0.85	89.3 89.3	0.343 0.343	
12	Back Base	28.5 36	19 24	0.85 0.58	89.3 104.7	0.343 0.292	

Wall : Height of wall : 16.40 ft
Top width of wall : 1.64 ft
Base width of wall : 6.35 ft
Unit weight of wall : 150 pcf

Ground motion:
α_h = 0.1, 0.2, 0.3, 0.4, 0.5
f_e = 1Hz, 2Hz

Notations:
ϕ = friction angle of the soil
δ = wall-soil friction angle
e = void ratio of the soil
γ = unit weight of soil
ν = Poisson's ratio of the soil

* 1kN/m^3 = 6.36 pcf

2. The parameters used in the analysis are shown in Table 2.

Main Program (WALL.FOR).

1. This program computes dynamic displacement by using the information generated by WALLINP.FOR.

2. Computed displacement includes sliding and rocking displacement occurring simultaneously. The displacement is recorded at each time step.

3. Calculate displacement based on non-linear soil properties including:

a. Shear strain vs. G/G_{max} for different soil types (Figure 4). The curve of gravel, sand, clay (P.I.=30) are used in the displacement analysis. The curve of silt is assumed as the mean value of sand and clay. Appropriate equations were used for G/G_{max} versus γ (Rafnsson 1991).

b. Shear strain vs. damping ratio for different soil types(Figure 5). The curve of silt is obtained from the mean value of sand and clay.

c. Dynamic stiffness in rocking and sliding (Gazetas and Tassoulas, 1987a).

d. Dynamic material and geometry damping (Gazetas and Tassoulas, 1987b).

4. Ground motion is sinusoidal.

5. The wall is sufficiently long for the end effects to be neglected.

6. The wall moves away from the backfill (active condition) when the earthquake acceleration is toward the backfill, and it moves toward the backfill (passive condition) when the earthquake acceleration is away from the backfill.

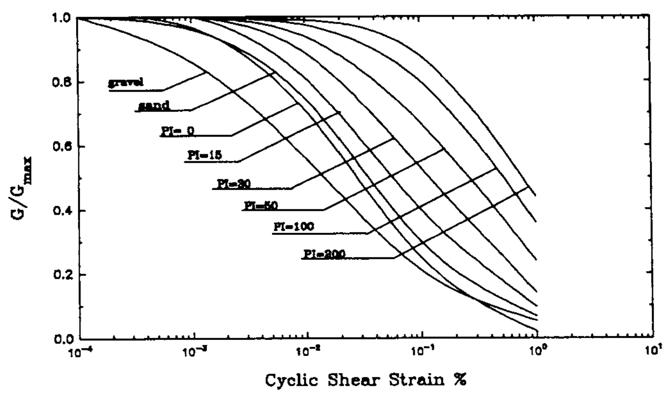


Figure 4. Average values of G/G_{max} versus shear strain (γ) for different types of soil (After Seed and Idriss 1970, for sand; Seed, Wong, Idriss and Tokimatsu 1986, for gravel; Vucetic and Dobry 1991, for PI=0-200)

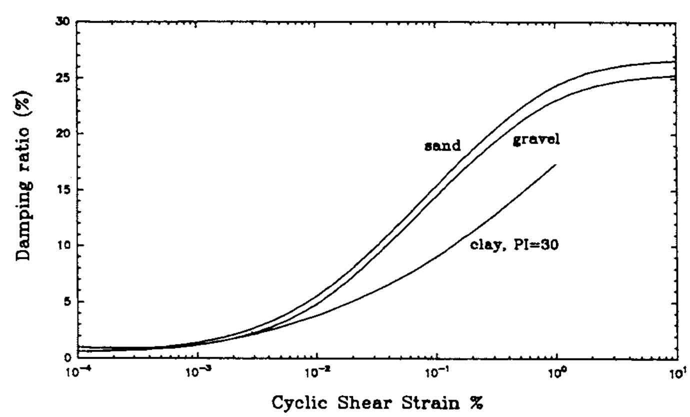


Figure 5. Average values of damping ratio (ζ) and shear strain (γ) for gravel, sand and clay (PI = 30) (After Seed and Idriss 1970, for sand; Seed, Wong, Idriss and Tokimatsu 1986, for gravel; Vucetic and Dobry 1991, for PI=30).

7. The wall displaces in a certain direction until the velocity of the wall motion becomes zero and reverses its direction.

8. The wall is assumed to rotate about its heel.

9. The mass of the backfill material participating in the wall motion is neglected.

10. Compare the computed displacement with the permissible displacement and calculate the new displacement by appropriately modifying the base width.

Check Program (WALLCHK.FOR).

Checking the factor of safety under dynamic loading, use the base width generated by the main program with the displacement within the permissible displacement (2% of the height of the wall).

The fourteen cases analyzed by Rafnsson (1991) are described by Rafnsson and Prakash (1994) and will not be repeated here.

PROPOSED WORK

The above analysis model was modified to compute the dynamic displacement of retaining walls for different soil combinations. Design charts have been developed for use by the field engineer (Prakash, Wu et al., 1995). Design engineers can also use those design charts to optimize their design work. The modifications to the program are as follows:

Input Program (WIP.FOR).

1. The program was developed based on the program named "WALLINP.FOR" proposed by Rafnsson (1991).

2. The base width used in the analysis is generated based on required factors of safety (Fang, 1991) under static loading:
- sliding ≥ 1.5
- overturning ≥ 1.5
- bearing capacity ≥ 2.5(added in the program)
- eccentricity ≤ B/6(added in the program)

For complete details, see Wu(1995).

Displacement Analysis (Main Program WAL.FOR).

1. This program was developed based on the "WALL.FOR" proposed by Rafnsson (Rafnsson, 1991).

a. Seven different types of base soil from well-graded gravel to low plastic silt had been chosen (Table 3).

b. Three cohesionless back fills had been used in the analysis (Table 3).

2. The computed base width and the input information can be generated automatically and used in the main program after the input program is executed.

3. The desired base width rather than the computed base width needs to be modified in the data file of the main program.

4. The subroutine in the main program calculating G_{max} has been adjusted. The Hardin and Black (1968, 1969) expression for maximum shear modulus (G_{max} - KN/m^2) by the following equation has been used:

$$G_{max} = 3230\ OCR^{k}\ \frac{(2.973-e)^2}{1+e}\ \sqrt{\bar{\sigma}_o} \qquad (5)$$

Where OCR : overconsolidation ratio
e : void ratio
$\bar{\sigma}_o$: mean effective pressure (KN/m^2)

Table 3. Engineering properties for both base soil and backfill.

BASESOIL (BS)

	soil type	γ_d KN/m^2	ϕ deg	δ deg	void ratio	ν	c KN/m^2	PI	w%	G_{max} Mpa
BS 1	GW	21.07	37.5	25.0	0.25	0.3	-	-	6	191
BS 2	GP	19.18	36.0	24.0	0.36	0.3	-	-	6	162
BS 3	SW	18.00	35.0	23.3	0.46	0.3	-	-	8	139
BS 4	SP	16.82	34.0	22.7	0.56	0.3	-	-	10	120
BS 5	SM	16.51	33.0	22.0	0.68	0.3	-	4	11	164
BS 6	SC	15.25	30.0	20.0	0.95	0.3	-	13	14	152
BS 7	ML	14.15	32.0	21.3	0.85	0.3	9.57	4	14	102

BACKFILL (BF)

	soil type	γ_d KN/m^2	ϕ deg	δ deg	void ratio	ν	c KN/m^2	PI	w%	G_{max} Mpa
BF 1	GM	19.6	33.0	22.0	0.35	0.3	-	-	10	164
BF 2	GP	18.9	34.0	22.7	0.40	0.3	-	-	8	153
BF 3	SP	15.6	34.0	22.7	0.50	0.3	-	-	8	132

* All properties for backfill are for the condition of 90 percent of the "Standard Proctor" maximum density

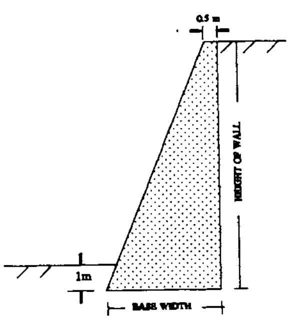
An equation expressing the relation of PI and k which is used to compute G_{max} has been added in the main program. The factor k used to calculate G_{max} depends on the plasticity index (PI) (Table 4). Equation 6 represents the mathematical expression between the value of k, and plasticity index (PI), and the computed data.

Table 4. Values of k (Hardin and Black, 1969).

PI	k
0	0
20	0.18
40	0.30
60	0.41
80	0.48
>100	0.50

$$k = -7.8*10^{-9}*PI^{4} + 1.5*10^{-6}*PI^{3} - 1.3*10^{4}*PI^{2} + 0.01*PI + 4.0*10^{-4} \quad (6)$$

5. A typical illustration of wall dimension, base soil and back fill conditions is shown in Figure 6. The 1 m embedded depth is used to calculate bearing capacity only. The resistance force due to the embedded soil is neglected. Therefore, the model is still treated as resting on the surface of the soil and the results will be conservative.



BF3-BS2
compacted poorly graded sand (SP)
ϕ = 34.0°
δ = 22.7°
γ = 16.8 KN/M^3
e = 0.50
ν = 0.3
w% = 8%
γ_c = 23.58 KN/M^3

Poorly graded gravel(GP)
ϕ = 36.0°
δ = 24.0°
γ = 20.33KN/M^3
e = 0.36
ν = 0.3
w_n% = 6%

STATIC FACTOR OF SAFETY
SLIDING ≥ 1.5
OVERTURNING ≥ 1.5
ECCENTRICITY ≤ B/6
BEARING CAPACITY ≥ 2.5

Figure 6. Retaining wall BF3-BS2.

Range of Parameters

The following is the range of parameters used in this study to develop design charts (Table 5).

The number of cycles used in the program is shown below (Table 6).

Table 5. Range of parameters used in the analysis

Parameters	Range
Height of wall	4,5,6,7,8,9,10m
Back fill	3 cases*
Base soil	7 cases*
Horizontal acceleration	0.1,0.2,0.3, 0.4,0.5g
Frequency of ground motion	1 Hz

*See Table 3.

Table 6. Number of cycles representative of different magnitude earthquakes (Seed et al., 1986).

Magnitude	N_c at $0.65\tau_{max}$
5.25	2-3*
6	5-6*
6.75	10
7.5	15
8.5	26

* Higher value is used.

RESULTS

Design Data

In the main program, the displacements at each time step are obtained from an acceptable difference (2%) between an assumed displacement and a computed displacement (Rafnsson, 1991). The displacement for each combination of seven base soils and three backfills was found at different heights of a wall (4, 5, 6, 7, 8, 9 and 10m), ground motion (0.1, 0.2, 0.3, 0.4 and 0.5g), and earthquake magnitude (M5.25, 6, 6.75, 7.5 and 8.5). A total of 3675 combinations was used in the analysis. The displacement was generated at specified conditions which include:

(1) Displacement in "m" and rocking "degree" at each ground motion (0.1, 0.2, 0.3, 0.4 and 0.5g) and earthquake magnitude of 5.25, 6, 6.75, 7.5 and 8.5 using 10m high wall and 1 Hz (Figure 7 and Table 7).

(2) Displacement and rocking (degrees) at each wall height (4, 5, 6, 7, 8, 9 and 10m) and each ground motion (Table 8) using the same magnitude (7.5) and the same frequency (1Hz).

(3) The natural frequency was determined at 0.2g ground motion and each wall height (Figure 8). The natural frequency for every case and every wall height was within 1.5 Hz to 4 Hz.

(4) The displacement was determined at each height of wall and number of cycles for ground motion 0.2g and frequency 1 Hz (Figure 9).

Table 7. Dynamic displacement (rocking degree) for 10m high wall (BF3-BS2).

Hor. Acc.	Displacement (m) (degree of rocking)				
	M=5.25	M=6.0	M=6.75	M=7.5	M=8.5
0.1	0.0112 (0.04)	0.0384 (0.12)	0.1227 (0.40)	0.3002 (1.01)	0.6318 (2.13)
0.2	0.0414 (0.13)	0.1896 (0.63)	0.4515 (1.52)	0.7409 (2.51)	1.3439 (4.56)
0.3	0.1166 (0.38)	0.3987 (1.34)	0.7501 (2.54)	1.1638 (3.95)	2.0459 (6.95)
0.4	0.2213 (0.74)	0.5919 (2.00)	1.0408 (3.53)	1.5808 (5.37)	2.7430 (9.32)
0.5	0.3547 (1.19)	0.8203 (2.72)	1.3501 (4.58)	2.0167 (6.85)	3.4591 (11.72)

* Frequency = 1 Hz

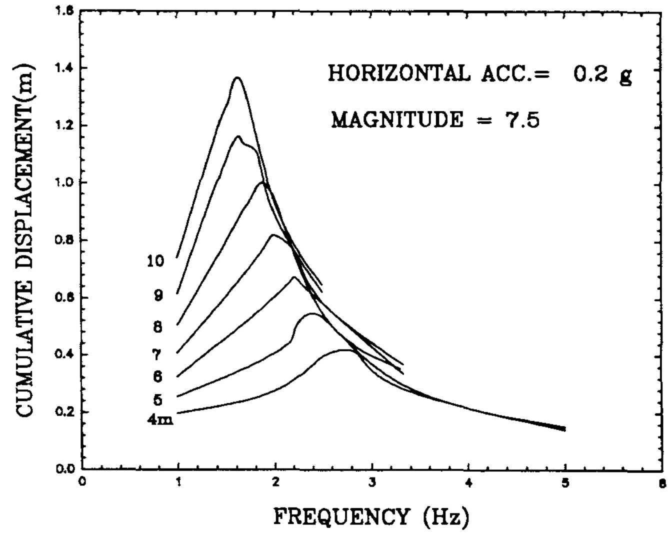


Figure 8. Cumulative displacement vs. frequency of ground motion for various wall heights (BF3-BS2).

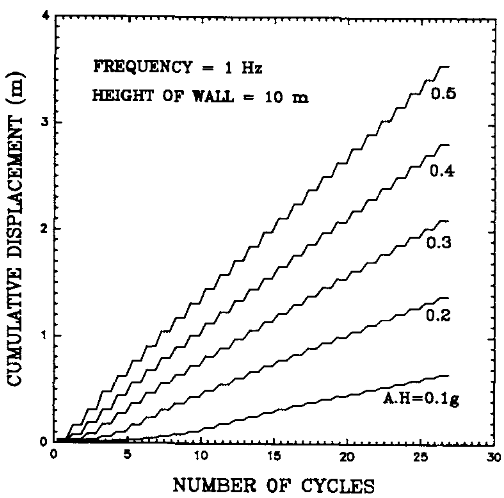


Figure 7. Cumulative displacement vs. number of cycles for different horizontal acceleration (BF3-BS2).

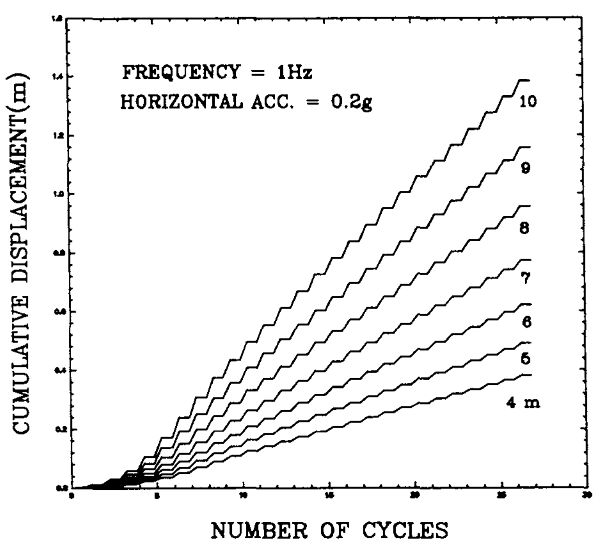


Figure 9. Cumulative displacement as a function of wall height (BF3-BS2)

Table 8. Dynamic displacement (rocking degree) for different wall heights (BF3-BS2).

Hor. Acc.	Displacement (m) (degree of Rocking)						
	4m	5m	6m	7m	8m	9m	10m
0.1	0.0728 (0.68)	0.0945 (0.67)	0.1219 (0.70)	0.1547 (0.75)	0.1966 (0.83)	0.2430 (0.91)	0.3002 (1.01)
0.2	0.1980 (1.85)	0.2556 (1.82)	0.3250 (1.88)	0.4081 (1.99)	0.5056 (2.14)	0.6166 (2.31)	0.7409 (2.51)
0.3	0.3181 (2.98)	0.4077 (2.92)	0.5164 (2.99)	0.6476 (3.17)	0.8045 (3.41)	0.9731 (3.66)	1.1638 (3.95)
0.4	0.4349 (4.08)	0.5578 (3.99)	0.7080 (4.11)	0.8884 (4.35)	1.0978 (4.67)	1.3241 (4.99)	1.5808 (5.37)
0.5	0.5338 (5.20)	0.7105 (5.09)	0.8982 (5.22)	1.1173 (5.47)	1.3796 (5.87)	1.6950 (6.39)	2.0167 (6.85)

* Magnitude = 7.5, Frequency = 1Hz

Design Charts.

1. Charts (Tables 9, 10) were prepared using the result from the main program for each case. The charts give the allowable horizontal acceleration for two critical conditions which include:
   (1) displacement within **5%** of wall height (Table 9).
   (2) displacement within **10%** of wall height (Table 10).

2. Tables 9 and 10 were prepared to show allowable horizontal acceleration using 1Hz frequency and 7.5 earthquake magnitude for various soil combinations.

Table 9. (Design chart-1) Allowable horizontal acceleration based on the displacement within **5%** of wall height with different wall height.

case	H α_h(g)	4m	5m	6m	7m	8m	9m	10m
1A	*BF1-BS1	0.3+	0.3-	0.3-	0.3-	0.3-	0.2+	0.2+
	BF2-BS1	0.3-	0.3-	0.3-	0.2+	0.2+	0.2+	0.2+
	BF3-BS1	0.2+	0.2+	0.2+	0.2+	0.2+	0.2-	0.2-
1B	BF1-BS2	0.3-	0.3-	0.2+	0.2+	0.2+	0.2-	0.2-
	BF2-BS2	0.3-	0.2+	0.2+	0.2-	0.2-	0.2-	0.2-
	BF3-BS2	0.2+	0.2-	0.2-	0.2-	0.2-	0.2-	0.1+
2A	BF1-BS3	0.4-	0.3+	0.3+	0.2+	0.3-	0.3-	0.2+
	BF2-BS3	0.4-	0.3+	0.3-	0.2+	0.3-	0.2+	0.2+
	BF3-BS3	0.3+	0.3-	0.3-	0.2-	0.2+	0.2+	0.2-
2B	BF1-BS4	0.3+	0.3-	0.3-	0.2+	0.2+	0.2+	0.2+
	BF2-BS4	0.3-	0.3-	0.3-	0.2-	0.2+	0.2+	0.2-
	BF3-BS4	0.3-	0.3-	0.2+	0.2-	0.2-	0.2-	0.2-
3A	BF1-BS5	0.3-	0.2+	0.2+	0.2-	0.2-	0.2-	0.2-
	BF2-BS5	0.3-	0.2+	0.2+	0.2-	0.2-	0.2-	0.2-
	BF3-BS5	0.2+	0.2-	0.2-	0.1+	0.2-	0.1+	0.1+
3B	BF1-BS6	0.2+	0.2-	0.2-	0.1+	0.1+	0.1+	0.1+
	BF2-BS6	0.2-	0.2-	0.1+	0.1-	0.1+	0.1+	0.1-
	BF3-BS6	0.2-	0.1+	0.1+	0.1+	0.1+	0.1+	0.1-
2c	BF1-BS7	0.3-	0.3-	0.3-	0.2-	0.2+	0.2+	0.2-
	BF2-BS7	0.2+	0.2+	0.2+	0.2-	0.2+	0.2-	0.2-
	BF3-BS7	0.2-	0.2-	0.2-	0.2-	0.2-	0.2-	0.2-

* combination of Backfill- Basesoil (Table 3)
earthquake magnitude = 7.5
frequency = 1Hz

Table 10. (Design chart-2) Allowable horizontal acceleration based on the displacement within **10%** of wall height with different wall height.

case	H α_h(g)	4m	5m	6m	7m	8m	9m	10m
1A	*BF1-BS1	0.5+	0.5+	0.5-	0.5-	0.4+	0.4+	0.4-
	BF2-BS1	0.5+	0.5-	0.4+	0.4+	0.4+	0.4-	0.4-
	BF3-BS1	0.4+	0.4+	0.4-	0.4-	0.4-	0.3+	0.3+
1B	BF1-BS2	0.5+	0.5-	0.4+	0.4+	0.4-	0.4-	0.4-
	BF2-BS2	0.5-	0.4+	0.4+	0.4-	0.3+	0.3+	0.3+
	BF3-BS2	0.4+	0.4-	0.3+	0.3+	0.3-	0.3-	0.3-
2A	BF1-BS3	0.5+	0.5+	0.5+	0.5+	0.5-	0.5-	0.4+
	BF2-BS3	0.5+	0.5+	0.5+	0.5-	0.5-	0.4+	0.4+
	BF3-BS3	0.5+	0.5-	0.5-	0.4+	0.4+	0.4-	0.4-
2B	BF1-BS4	0.5+	0.5+	0.5-	0.4+	0.4+	0.4-	0.4-
	BF2-BS4	0.5+	0.5-	0.4+	0.4+	0.4-	0.4-	0.3+
	BF3-BS4	0.4+	0.4+	0.4-	0.4-	0.3+	0.3+	0.3+
3A	BF1-BS5	0.5-	0.4-	0.4-	0.4-	0.3+	0.3+	0.3-
	BF2-BS5	0.4+	0.4-	0.4-	0.3+	0.3+	0.3-	0.3-
	BF3-BS5	0.4-	0.3+	0.3+	0.3-	0.3-	0.3-	0.2+
3B	BF1-BS6	0.3+	0.3-	0.3-	0.2+	0.2+	0.2-	0.2-
	BF2-BS6	0.3-	0.3-	0.2+	0.2+	0.2+	0.2-	0.2-
	BF3-BS6	0.3-	0.2+	0.2+	0.2+	0.2+	0.2-	0.2-
2C	BF1-BS7	0.5-	0.5-	0.4+	0.4+	0.4+	0.4-	0.4-
	BF2-BS7	0.4+	0.4+	0.4+	0.4+	0.4-	0.4-	0.3+
	BF3-BS7	0.3+	0.3+	0.3+	0.3+	0.3+	0.3+	0.3-

* combination of Backfill-Basesoil (Table 3)
earthquake magnitude = 7.5
frequency = 1Hz

3. Each chart was divided to several cases:
   (1) gravelly base soil,
   (2) sandy and silty base soil,
   (3) clayey and silty sand base soil,

4. The numbers shown in Tables 9 and 10 are the allowable ground acceleration with different heights of wall. The plus sign shows the allowable ground acceleration is higher than this number and the displacement at this ground acceleration is smaller than 5% or 10% of wall height. That means the allowable displacement will be developed at higher ground motion than the number indicated in the table. The minus sign shows the allowable acceleration will be smaller than the number indicated in the tables.

5. The practicing engineers can use tables to determine the allowable horizontal acceleration for desired soil combinations. For computation details, see Wu and Prakash (1995), Wu and Rafnsson (1995).

Design Equation.

The results of displacement computations for each combination were developed into three cases. The three cases include:
(1) gravelly base soil and three back fills (cases 1A, 1B, Tables 9 and 10),
(2) sandy and silty base soil and three back fills (cases 2A, 2B, 2C, Tables 9,10),
(3) Clayey and silty sand base soil and three back fills (cases 3A, 3B, Tables 9,10).

Each case was used to find an equation to calculate displacement instead of using the computer program. The nonlinear estimation method was used to find equations. The following cases show the equations and computed results.

The computed variations for each of the equations are shown elsewhere (Wu 1995).

Case 1: This case is used for gravelly base soil and three back fills (1A, 1B).

$$D = c1*\left(\frac{B}{H}\right)^{c2}*\left(\frac{\gamma_b}{\gamma_f}\right)^{c3}*\left(\frac{\tan\phi_b}{\tan\phi_f}\right)^{c4}*(n)^{c5}*(\alpha_h)^{c6} \tag{7}$$

c1:	146.339292	c4:	-14.107555
c2:	9.363453	c5:	1.140192
c3:	11.681722	c6:	1.135677

The R-SQUARED (1- residual/total) = 0.99

Where
D : displacement (m)
γ_b, γ_f : the bulk density(KN/m^3) of base or fill
ϕ_b, ϕ_f : the friction angle (degree) of base or fill.
B : base width
α_h : ground motion (0.1g - 0.5g).
H : height of wall (m).
n : number of cycles.

Case 2: This case is used for sandy and silty base soil and three back fills(2A, 2B, 2C).

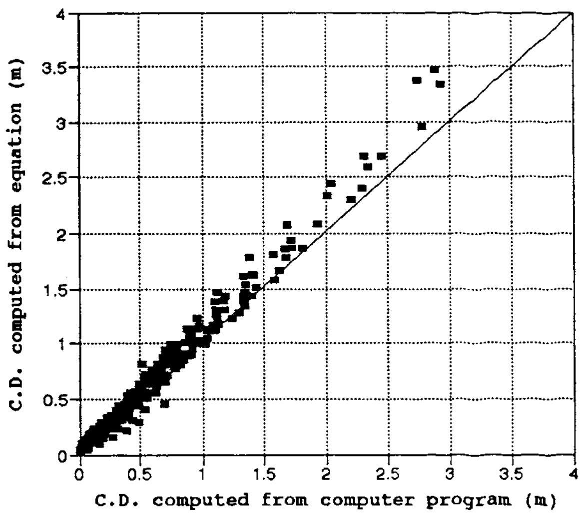
$$D = c1*\left(\frac{B}{H}\right)^{c2}*\left(\frac{\gamma_b}{\gamma_f}\right)^{c3}*\left(\frac{\tan\phi_b}{\tan\phi_f}\right)^{c4}*(n)^{c5}*(\alpha_h)^{c6}*(C)^{c7} \tag{8}$$

c1: 5.523691 c4: -8.311921 c7: 0.491983
c2: 6.013044 c5: 1.196699
c3: 7.645120 c6: 1.147220
The R-SQUARED (1-residual/total)=0.97

Case 3: This case is used for low strength base soil and three back fills (3A, 3B).

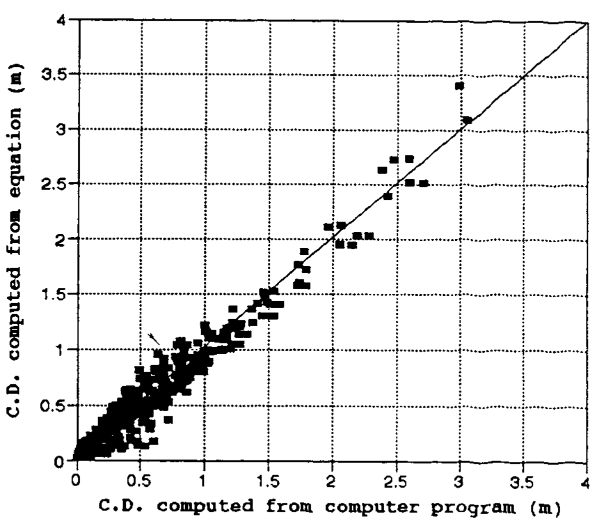
$$D = c1*\left(\frac{B}{H}\right)^{c2}*\left(\frac{\gamma_b}{\gamma_f}\right)^{c3}*\left(\frac{\tan\phi_b}{\tan\phi_f}\right)^{c4}*(n)^{c5}*(\alpha_h)^{c6} \quad (9)$$

c1: 0.232841 c4: -1.454632
c2: 2.156554 c5: 1.391014
c3: 1.455370 c6: 1.126569
The R-SQUARED (1-residual/total) = 0.92



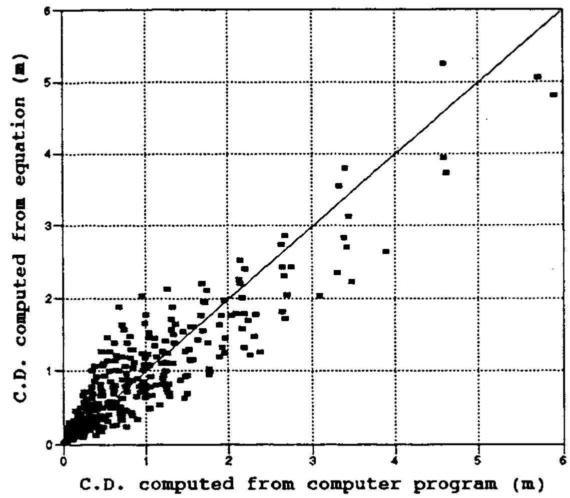
C.D. = Cumulative displacement (m).

Figure 10. Diagram showing the variation between values computed by the computer program and the nonlinear estimation method (case 1).



C.D. = Cumulative displacement (m).

Figure 11. Diagram showing the variation between values computed by the computer program and the nonlinear estimation method (case 2).



C.D. = Cumulative displacement (m).

Figure 12. Diagram showing the variation between values computed by the computer program and the nonlinear estimation method (case 3).

The value of R-SQUARED shows the accuracy of estimation. The value 1 indicates the estimation is precise. Figures 10 (case 1), 11 (case 2), and 12 (case 3) show the correlation of the actual results computed by the program and values computed by the equation. If the value computed by the equation is equal to the computer value, it will plot on the 45° line. Thus, the closer the points plot to this line the better the fit of the equation. These equations apply to an exciting frequency of 1 Hz. The natural frequency cannot be computed by the equations. It can been seen from Figures 10, 11 and 12 that the equations yield essentially the same result as the computer program. Thus, the designer may use equations 7, 8 and 9 instead of the computer program for situations closely approximating the conditions used in developing the equations.

CONCLUSIONS

A modified analysis model has been generated to obtain more appropriate earthquake-induced displacement of rigid retaining walls using non-linear behavior of both back fill and base soil. Twenty-one different types of base soil, backfill, five different ground motions, seven heights and 5 earthquake magnitudes, which gave 3675 combinations for analysis, have been studied in this research. The following conclusions have been drawn from this research.

1. The model gives displacements for simultaneous sliding and rocking for all base soils and backfills considered. This is an advancement over other similar models. (Nadim and Whitman 1984 and Rafnsson and Prakash 1991).

2. Two critical conditions were used to define the allowable displacement in the analysis, i.e., 5% or 10% of wall height. For specified parameters, charts have been developed based on the permissible displacement. These design tables help practical designers to find the allowable ground motion and optimize their design work.

3. Three equations (cases 1, 2 and 3) for three base soil categories have been developed to predict the displacement instead of using the computer program. The variations between the equations and computer program for case 1, case 2 and case 3 are less than 8%. The results of these three equations are acceptable.

4. At this stage, we do not know what is acceptable displacement of a wall. Therefore, the profession needs to pay attention to this question.

ACKNOWLEDGEMENT

The draft was commented upon by Sanjeev Kumar, Shahbaz Muhammad Chaudhry and Dr. George M.S. Manyando.

REFERENCES

Amana, R., H. Azuma, Y. Ishii (1956), "Aseismic Design of Quay-walls in Japan", Proceedings of the First World Conference on Earthquake Engineering, Berkeley, CA, June, pp. 32-1 to 32-17.

Fang, H.Y. (1991), "Foundation Engineering Handbook", 2nd Ed., Van Nostrand Reinhold, Inc, NY, pp. 487-491.

Gazetas, G. and J.L. Tassoulas (1987a), "Horizontal Stiffness of Arbitrarily Shaped Embedded Foundations", Journal of Geotechnical Engineering, Vol. 113, No. 5, May, pp. 440-457.

Gazetas, G. and J.L. Tassoulas (1987b), "Horizontal Damping of Arbitrarily Shaped Embedded Foundations", Journal of Geotechnical Engineering, Vol. 113, No. 5, May, pp. 458-475.

Hardin, B.O. and W.L. Black (1968), "Vibration Modulus of Normally Consolidated Clay", Journal of the Soil Mechanics and Foundations Division, Vol. 94, No. SM2, March, pp. 353-369.

Hardin, B.O. and W.L. Black (1969), "Vibration Modulus of Normally Consolidated Clay; Closure", Journal of the Soil Mechanics and Foundations Division, Vol. 95, No. SM6, Nov., pp. 1531-1537.

Mononobe, N. and H. Matsuo (1929), "On the Determination of Earth Pressure During Earthquakes", World Engineering Congress Proceedings, Vol. IX, Tokyo, pp. 177-185.

Nadim, F. and R.V. Whitman (1984), "Coupled Sliding and Tilting of Gravity Retaining Walls During Earthquakes", Proceedings of the Eighth World Conference on Earthquake Engineering, July 21-28, San Francisco, CA, Vol. III, pp. 477-484.

Nandakumaran, P. (1973),"Behavior of Retaining Wall Under Dynamic Loads", PhD Thesis, University of Roorkee, Roorkee (UP), India.

Okabe,S. (1926), "General Theory of Earth Pressure," Journal of the Japanese Society of Civil Engineers, Tokyo, Japan, Vol. 12, No. 1.

Prakash, S. (1981a), "Soil Dynamics" McGRAW-HILL Book Co., New York, N.Y., Reprint, Shamsher Prakash Foundation, Rolla MO.

Prakash, S. (1981b), "Analysis of Rigid Retaining Walls During Earthquakes", Proc. International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics", St. Louis, MO, April 26 - May 3 pp. 993-1020.

Prakash, S., Y. Wu and E. A. Rafnsson (1995), "Displacement Based Aseismic Design Charts For Rigid Retaining Walls", Shamsher Prakash Foundation, Rolla MO.

Rafnsson, E.A. (1991), "Displacement Based Design of Rigid Retaining Walls Subjected to Dynamic Loads Considering Soil Nonlinearity", PhD Thesis, University of Missouri-Rolla, USA.

Rafnsson, E.A. and S. Prakash (1991), "Stiffness and Damping Parameters for Dynamic Analysis of Retaining Walls", Proc. Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. III, pp. 1943-1952.

Rafnsson, E.A. and S. Prakash (1994), "Displacement Based Aseismic Design of Retaining walls", Proc. XIII Inter. Conf. on SMFE, New Delhi, Vol 3, pp. 1029-1032.

Richards, R. and D.G. Elms (1979), "Seismic Behavior of Gravity Retaining Walls", Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT4, April, pp. 449-464.

Seed, H.B. and I.M. Idriss (1970), "Soil Moduli and Damping Factors for Dynamic Response Analysis", Report No.EERC 70-10, Earthquake Engineering Research Center, Dec., pp 1-40.

Seed, H.B. I.M. Idriss and I. Arango (1983), "Evaluation of Liquefication Potential Using Field Performance Data", Journal of Geotechnical Engineering, Vol. 109, No. 3, March, pp. 458-482.

Seed, H.B., R.T. Wong, I.M. Idriss and K. Tokimatsu (1986), "Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils", Journal of Geotechnical Engineering, Vol. 112, No. 11, November, pp. 1016-1032.

Shakya, D.A. (1987), "Engineering Solutions for the Displacement of Rigid Retaining Walls Subjected to Earthquake Loads", M.Sc. Thesis Presented to UMR.

Vucetic, M. and R. Dobry (1991), "Effect of Soil Plasticity on Cyclic Response", Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 1, January, pp. 89-107.

Wu, Y. (1995), "Displacement Based Seismic Design Charts for Rigid Retaining Walls", M.S. Thesis, Univ. of Missouri-Rolla.