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# Analysis of Retaining Wall Subjected to Earthquake Loading

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## Abstract

A numerical method through finite element(FEM) with two models: Elastic &Equivalent Linear was used to investigate the seismic behavior of retaining wall supporting saturated, liquefiable, cohesionless backfill soil. Horizontal/Vertical displacement, pore water pressure, horizontal total stress in the soil at the face of the wall, and Max. shear stress in the soil at the base were measured. It was shown that the Equivalent model gives more reasonable results and the liquefaction zones concentrated in the passive side more than the active side. Max. horizontal displacement at the top of the wall reaches 0.67m while vertical displacement increased in the range(66-116)% with the wall increasing in dimensions. Both pore water pressure/horizontal total stress increased with time/dimensions in the range(37%),(200%) respectively.

## Introduction

Despite advances in geotechnical engineering, it is common to find retaining walls experiencing near or complete failure during strong earthquakes(Seed & Whitman,1970). Effect of earthquakes on retaining walls often include large translation and rotational displacements, buckled walls, settlement of backfill soils, and failure of structures found on the backfill. Excessive displacement cannot only induce failure of the wall itself but may also cause damage to structures nearby(Zeng & Steedman, 2000).

Damage to retaining walls can be great,due to an incomplete understanding of the complex soil-structure interaction occurring during an earthquake.

The magnitude and distribution of additional, seismic, lateral earth pressures are particularly in question(Mandar & Ronald, 2001). Seismic behavior of a retaining wall/soil system is a function of a backfill soil properties, relative stiffness of the wall/soil system, wall fixity conditions, foundation stability, and characteristics of applied earthquake motions. For a retaining wall with a dry backfill, the increase in lateral pressures, due to an earthquake, needs to be determined. If the backfill is saturated, the design is further complicated by the dynamic pore pressure that cause extra lateral load on retaining walls. In addition, excess pore pressures may develop with cyclic loading the result of which is the reduction of strength and stiffness of the backfill. The conditions become worse if the soil liquefies and loses all of its shear strength(Mandar& Ronald, 2001).

The distribution of seismic pressure on retaining structures is basically a problem of soil-structure interaction. Because of incompatibility and in some situations, the discontinuity of the deformations in the near and far field, the problem becomes complicated(Rowland et al.,1999).

The Mononobe-Okabe (M-O) method (Mononobe & Matsuo 1929; Okabe, 1924), in its original or modified form, is used to estimate the seismic lateral thrust on the wall. This pseudostatic, limit equilibrium method is an extension of Coulomb's earth pressure theory and based on rigid plasticity. It was originally developed for rigid retaining walls with dry, cohesionless backfills. For saturated backfills, the M-O method is extended to incorporate hydrodynamic effects and permeability (Matsuzwa et al.1985). A modified M-O method for liquefiable backfills was assumed the soil had completely liquefied and acted as a heavy fluid.

Zhang et al.(1998) introduced a concept of "submerged effective unit weight" which accounted for an excess pore pressure ratio and a method to evaluate dynamic soil and water pressures on waterfront rigid walls under lateral wall/soil deformation. Numerical methods have proven to produce reasonable and realistic results for dynamic problems defining soil-structure interaction. One of these numerical method is the finite element method which has been used successfully to solve many problems dealing with soil structure interaction including footings, retaining structures, piles, underground structures, buildings and dams(Desai & Christian, 1977). Wood(1975) used FEM for studying the dynamic pressure against a fixed structure where the soil is considered as a uniform elastic material. Pitilakis and Moutsakis (1989) used FEM of the seismic response of a gravity quay wall where the results of wall displacement and ground settlement were compared with data recorded in the field. A study made on the effect of earthquake shaking on changing horizontal/vertical displacement, pore water pressure, and Max. shear stress at the base of the wall with time. Also the effect of the earthquake and the changing of the wall dimensions on the generated liquefaction zones around/under the wall was studied.

## The Finite Element Method of Analysis

The finite element method is an efficient numerical method to solve such problems in which a two-dimensional plain strain analysis of the soil-structure system can be considered. Appropriate values of soil properties can be included by selecting values that are compatible with the computed strains in the soil deposits (Elewi, 2003). The major points that used in FEM is described below:

**1.Finite Element Equations:**

Motion Equation: The governing motion equation for dynamic response of a system in finite element formulation can be expressed as(Bathe &Wilson,1976):

$$[M]\{\ddot{a}\}+[D]\{\dot{a}\}+[K]\{a\}=\{F\}\text{..........}(1)$$

Where : $[M]$ = mass matrix, $[D]$ = damping matrix, $[K]$ = stiffness matrix,

$\{F\}$ = vector of loads

$\{\ddot{a}\}$ = vector of nodal accelerations, $\{\dot{a}\}$ = vector of nodal velocities,

$\{a\}$ = vector of nodal displacements

$$\{F\}=\{F_b\}+\{F_s\}+\{F_n\}+\{F_g\}\text{..........}(2)$$

Where : $\{F_b\}$ = body force, $\{F_s\}$ = force due to surface boundary pressures,

$\{F_n\}$ = concentrated nodal force,

$\{F_g\}$ = force due to earthquake load.

The vector of loads could made up by different forces:

Mass Matrix [M]: The mass matrix named a lumped mass matrix which can be expressed as:

Damping matrix [D]: It is common practice to assume the damping matrix to be a linear combination of mass matrix and stiffness matrix:

Stiffness matrix [K]:

The stiffness matrix is:

$$[M]=\int_V \rho[\psi]dv\text{..........}(3)$$

Where : $\rho$ = mass density ,

$[\Psi]$ = a diagonal matrix of mass distribution factors.

$$[D]=\alpha[M]+\beta[K]\text{..........}(4)$$

$\alpha, \beta$ : are scalers called Rayleigh damping coefficients.

They can be related by damping ratio $\eta$ by : $\eta=\dfrac{\alpha+\beta\omega^2}{\omega}$

$\omega$ : Particular frequency of vibration for the system.

Body force: For a given material, the body force is calculated from the density of the material.

Force due to Boundary Stresses: It represents the nodal forces caused by

$$[K]=t\int_A [B]^T[C][B]dA\text{..........}(5)$$

Where : $[B]$ = strain - displacement matrix, $[C]$ = constitutive matrix,

t = constant element thickness.

externally applied pressure along the boundary of the element.

<u>Force due to earthquake load:</u>

$$\{Fg\} = [M]\{\ddot{a}\} \qquad (6)$$

$[M]$ = mass matrix, $\{\ddot{a}\}$ = applied nodal accelerations.

**2. Temporal Integration:**

The motion equation is a second-order propagation type of equation, which can be solved in either frequency domain or time domain. Solution in time is preferred when material property may change with time. Wilson-$\theta$ methods (Bathe & Wilson,1976) is used to perform the time domain integration of motion equation in which the displacement, velocity and acceleration at time t are known, the acceleration is assumed to be linear from time t to time $\theta\Delta t$, then the velocity and displacement at any time can be obtained by integrating the acceleration and velocity respectively.

## The Quake/W Program

The Quake/W program was used in this study which depends on FEM based on motion equation and having two constitutive models: linear-elastic model and equivalent linear model. The equivalent linear model is actually non-linear, but it is equivalent to a linear model because it transforms the irregular earthquake shaking into equivalent uniform cycles. It is non-linear in that the shear modulus G is modified (reduced) in response to cyclic shear strains (see Fig.(1)). Each iteration is linear (G is a constant), but the modification of G after each iteration makes the analysis non-linear. Isoparametric quadrilateral and triangular finite elements with no specific limits on problem size in terms of number of nodes, element or material types are used in Quake/W program because it depends on dynamic memory allocation.

## Case Study

In this study, four cases with two models (Linear Elastic, Equivalent Linear) were studied in which through Fig.(2) the wall dimensions can be seen. Table(1) shows the material properties for every model with their dimensions for four cases. Fig.(3) shows the FE mesh used in the analysis. Acceleration time history for El-Centro earthquake(Nadim& Whitman,1983) that needed for the analysis is shown in Fig.(4). The time steps is 0.02Sec and is applied through 500 steps. Fig.(5) shows the relation between cyclic shear strain& damping ratio (Kramer,1996).

## Results

For every case same parameters were studied. Table (2)contains figures for case(1) showing the liquefaction zones around and under the retaining wall at the end time of the earthquake shaking. The figure also shows the changing of the horizontal displacement with time during the earthquake at different nodes and for both linear elastic and equivalent linear models. The same figures for case (2),(3) and(4) are shown in Tables (3), (4) and (5) respectively. Tables (6),(7),(8) and (9) have figures for vertical displacement changing with time during earthquake shaking with figures showing the pore water pressure change with time at three selected nodes for cases(1),(2),(3) and (4) and for both models respectively. The final Table (10) includes figures for horizontal total stress change with elevation at the face of the retaining wall for four cases. The figures also represents the max shear stress change with time at the base of the retaining wall for cases(1) and (3) and for both models.

## Conclusions

For the studied cases with the given tables including the results, the following points can be concluded:

**1. Liquefaction zones:**

(a) These zones are concentrated in the passive side more than the active side which means that the earthquake has little effect on changing the effective stress in the active zone.

(b) The Equivalent Linear model gives more reasonable results due to actual represent of pore water pressure generation during earthquake and because of the reduced shear modulus (G).

(c) As the studied area increases with the increasing of the wall height/base dimensions, the liquefaction zones decrease due to the dissipation of earthquake intensity which lead to little effect on pore water pressure.

**2. Horizontal displacement:**

(a) Maximum displacement reaches 0.45m at the top of the wall for case(1) and this value is 0.5m, 0.67m & 0.5m for other cases respectively.

(b) Equivalent Linear model gives greater max. horizontal displacement at ratios of 170%,120%,150% & 150% respectively.

(c) For all the cases, the active zone beyond the wall (backfill soil) have maximum horizontal displacement.

(d) When the wall height/base increases the horizontal displacement increases in the range (10-35)%.

**3. Vertical displacement:**

(a) Max. vertical displacement occurs at the base of the wall and increased in the range(66-116)% for the studied cases.

(b) Vertical displacement estimated by Equivalent Linear model is greater than the Elastic model and may reach (170%).

(c) Little oscillation in vertical displacement happened when the height/base of wall increases.

**4. Pore Water Pressure (PWP):**

(a) The PWP increases with high speed to reach max. value at time (1)Sec. during earthquake shaking.

(b) Both models gave approximately the same pwp.

(c) With increasing wall dimensions, PWP increased due to increasing in the water table level in the range (37%).

**5. Horizontal Total Stress (HTS):** (At the end of the earthquake)

(a) HTS increases with increase in the elevation/time because the pwp reaches the maximum value (i.e. increase in both vertical & horizontal stress).

(b) With greater wall height used, greater HTS can be obtained (200%).

(c) Little oscillation in HTS was obtained for both models.

**6. Max. Shear Stress at the base of the wall:** (case1 and case 3 only)

(a) As earthquake acceleration changes with time, the max. shear stress oscillates under the base because this zone is nearly liquefied.

(b) The final max. shear stress for case(1) is greater than that of case(3) which means more PWP oscillation in case(3)(height/base is greater) and this leads to less normal stress (also less shear stress).

(c) Equivalent Linear model gives little change in max. shear stress with time.

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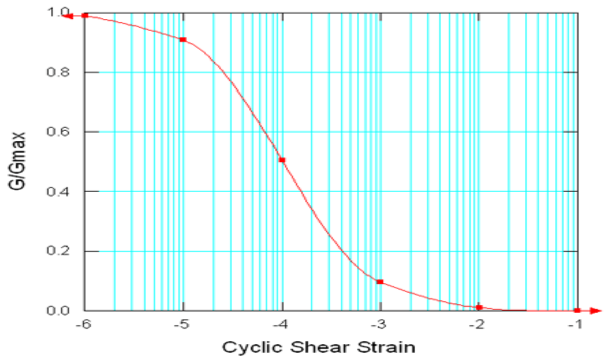


Figure (1)The relation between G& cyclic shear strain(Kramer,1996).

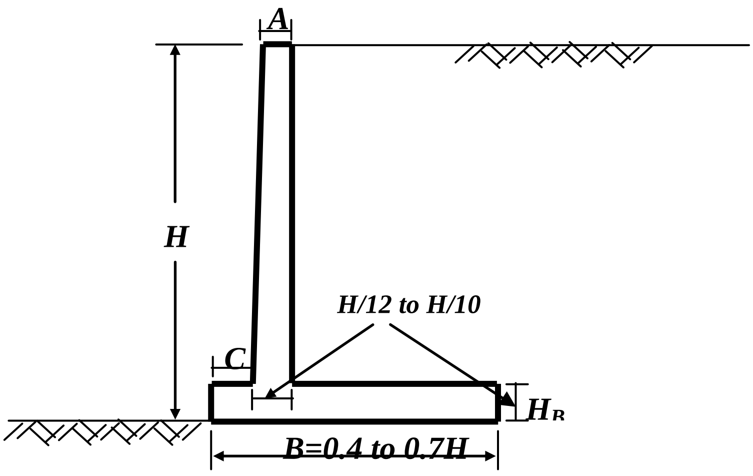


Figure (2) Typical cantilever retaining wall (Bowels, 1988).

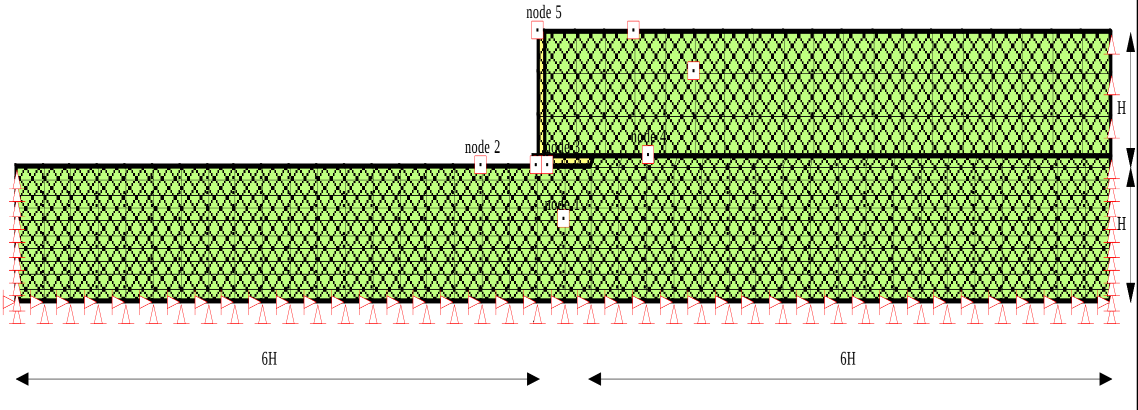


Figure (3) The finite element mesh.

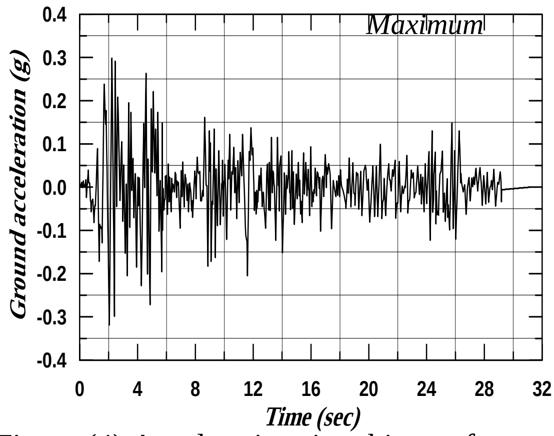



Figure (4) Acceleration time history for El-Centro earthquake (Nadim,1983).

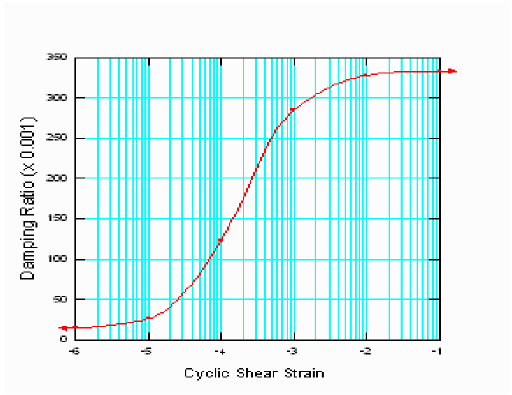
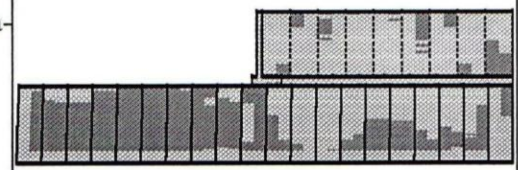
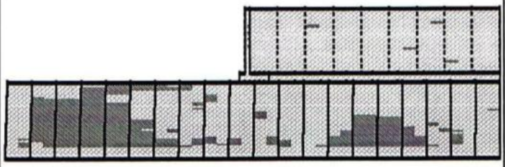
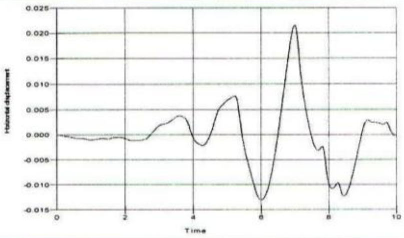
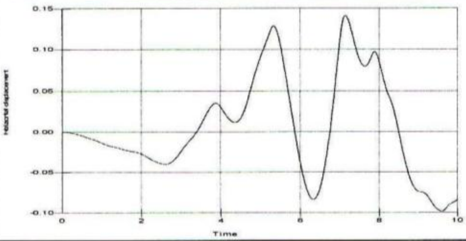
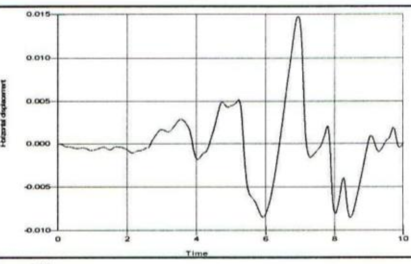
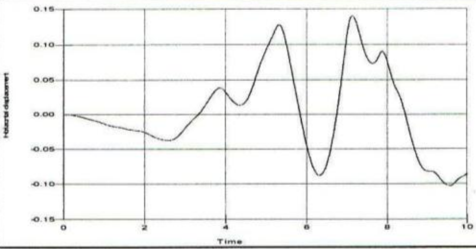
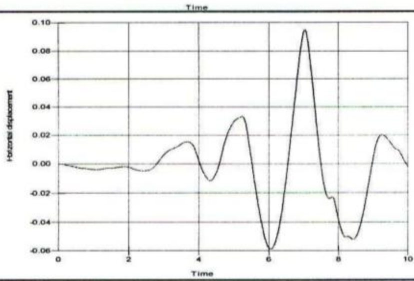
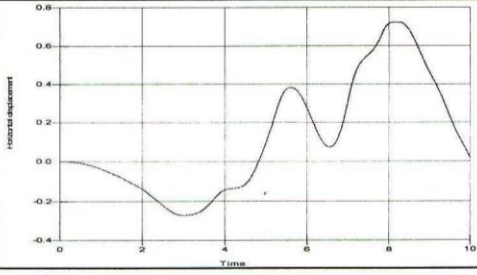
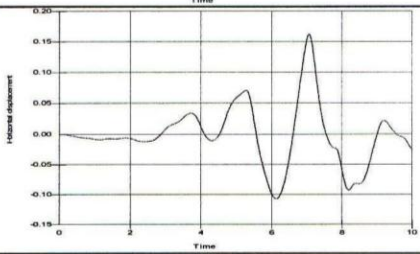
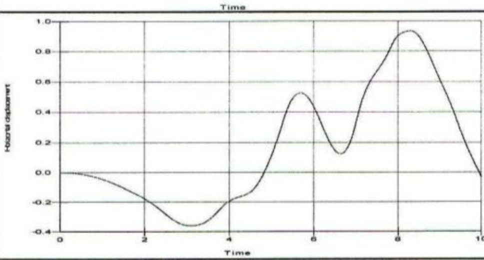
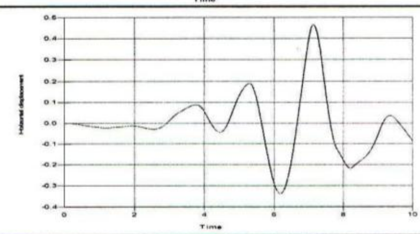
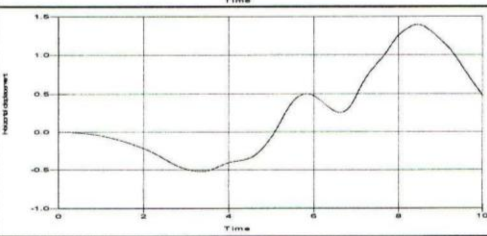



Fig.(5)The relation between cyclic shear strain &damping ration (Kramer,1996).

Table (1): Material Properties and Wall dimensions (Bowels, 1988)

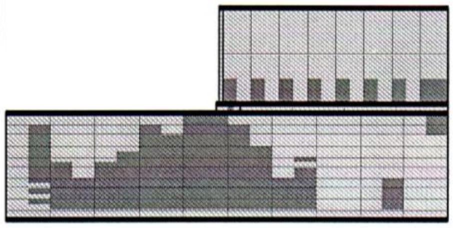
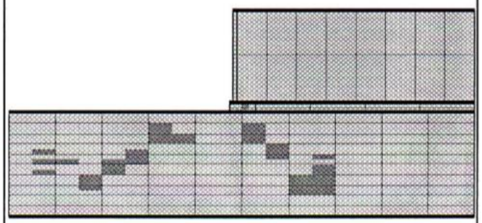
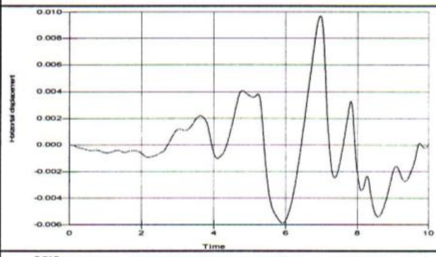
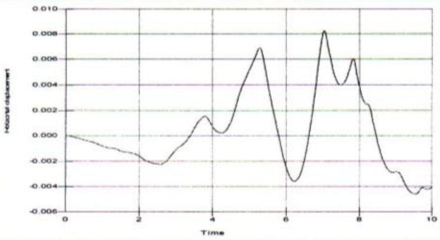
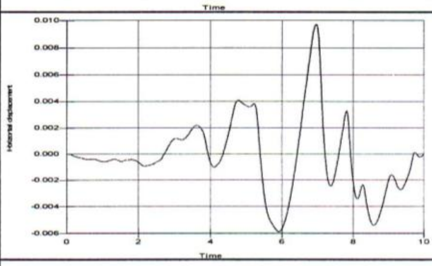
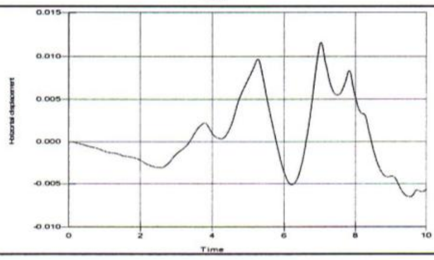
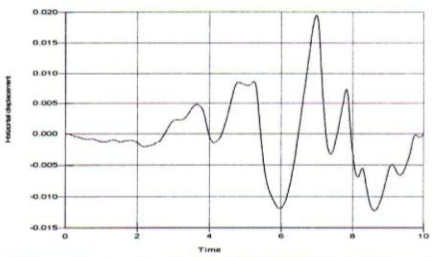
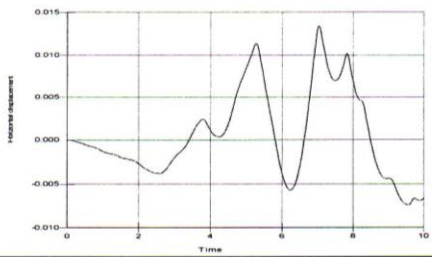
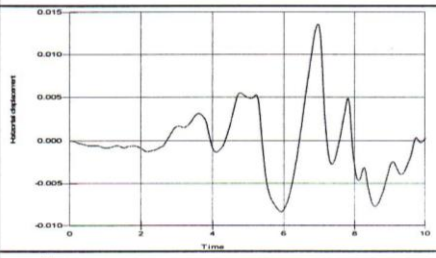
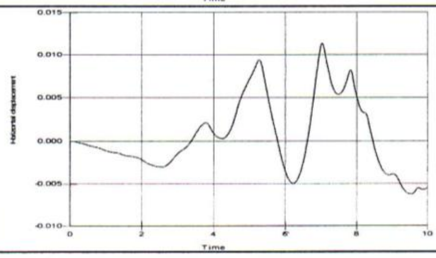
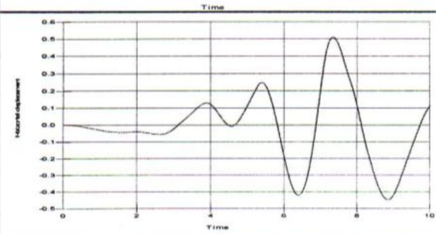
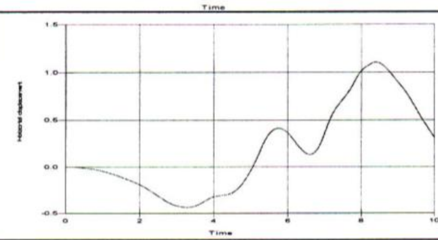
| | | Case1 | Case2 | Case3 | Case4 |
|---|---|---|---|---|---|
| | H(m) | 4 | 5 | 6 | 7 |
| | A(m) | 0.28 | 0.35 | 0.42 | 0.49 |
| | B(m) | 2.8 | 3.5 | 4.2 | 4.9 |
| | C(m) | 0.4 | 0.5 | 0.6 | 0.7 |
| | $H_B$(m) | 0.4 | 0.5 | 0.6 | 0.7 |
| | Properties | Wall | | Soil | |
| Elastic Model | Unit weight (kN/m$^3$) | 23.25 | | 17 | |
| | Young's modulus, E (kN/m$^2$) | 17384000 | | 11500 | |
| | Poisson's ratio, υ | 0.18 | | 0.2 | |
| Equivalent Linear Model | Damping ratio | - | | 0.2 | |
| | Poisson's ratio, υ | - | | 0.2 | |
| | Shear Modulus G(kN/m$^2$) | - | | 3550 | |

**Table (2)**

| Case 1 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Liquefaction zones |  |  |
| Node1 Horz. Disp. Versus time |  |  |
| Node2 Horz. Disp. Versus time |  |  |
| Node3 Horz. Disp. Versus time |  |  |
| Node4 Horz. Disp. Versus time |  |  |
| Node5 Horz. Disp. Versus time |  |  |

Note: Units: time=Sec., Horizontal displacement= meter.

**Table (3)**

| Case 2 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Liquefa-ction zones |  |  |
| Node1 Horz. Disp. Versus time |  |  |
| Node2 Horz. Disp. Versus time |  |  |
| Node3 Horz. Disp. Versus time |  |  |
| Node4 Horz. Disp. Versus time |  |  |
| Node5 Horz. Disp. Versus time |  |  |

Note: Units: time=Sec., Horizontal displacement= meter.

**Table (4)**

| Case 3 | Linear Elastic. | Equivalent Linear. |
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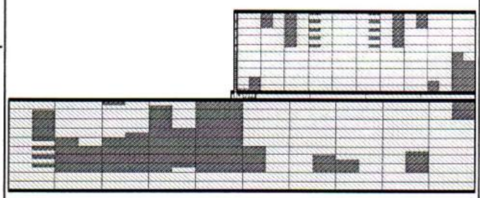
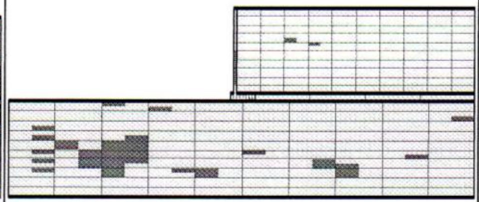
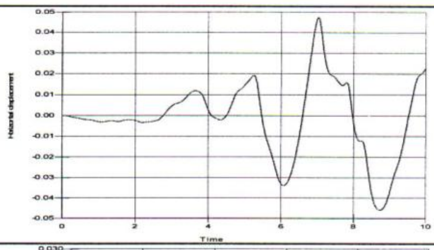
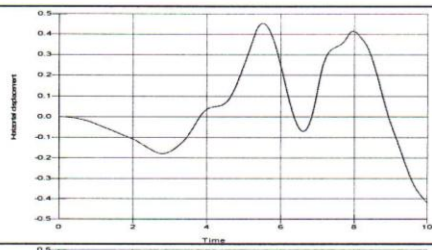
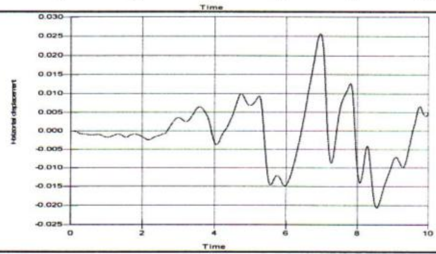
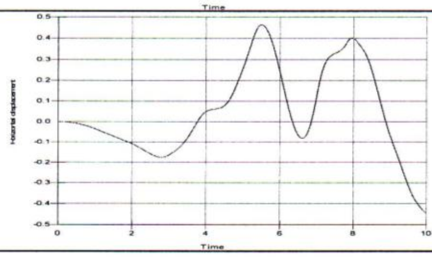
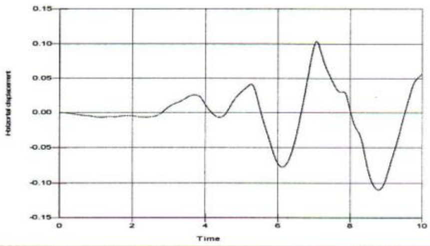
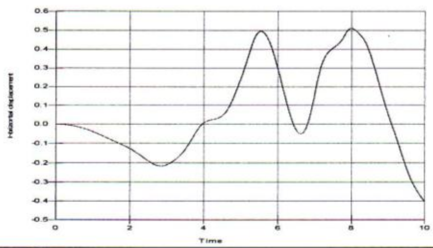
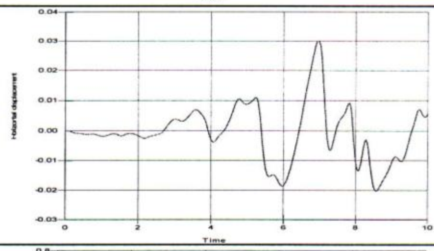
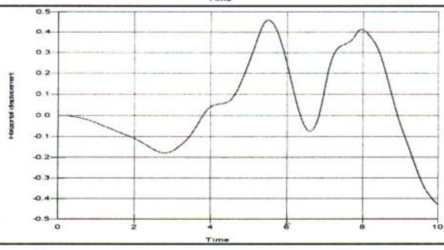
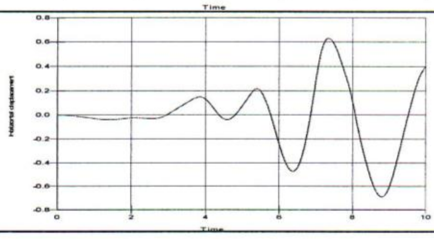
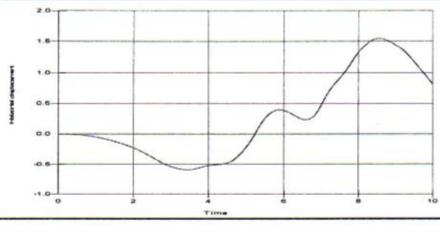
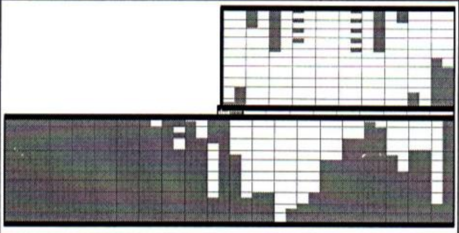
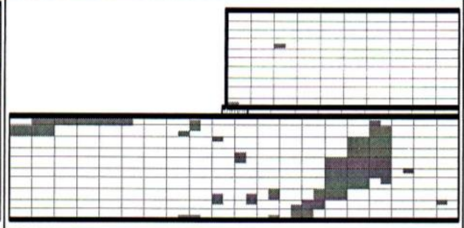
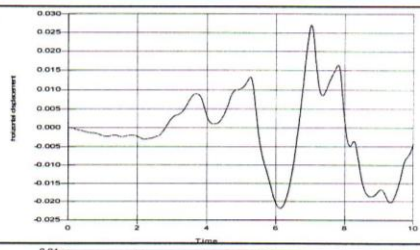
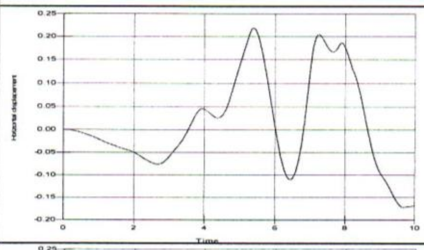
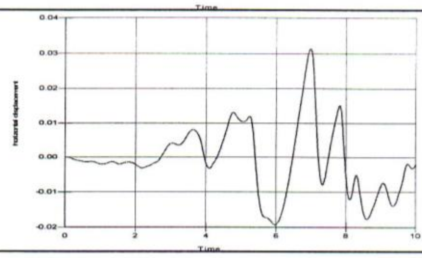
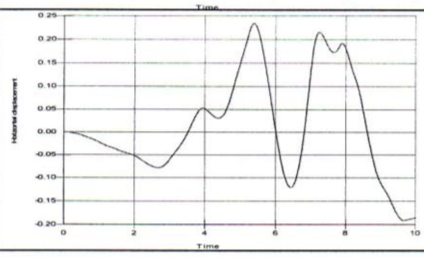
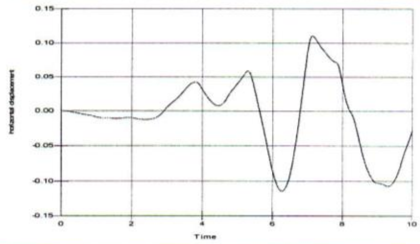
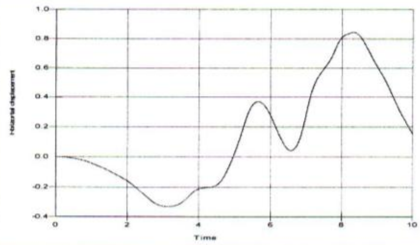
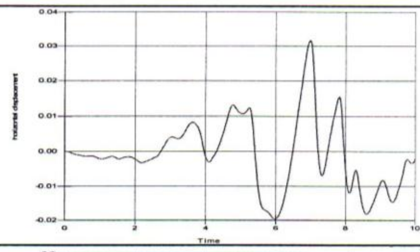
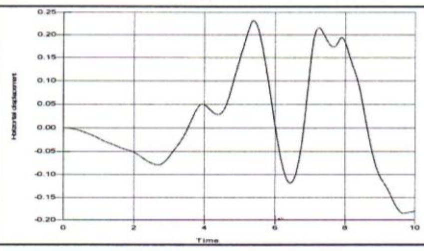
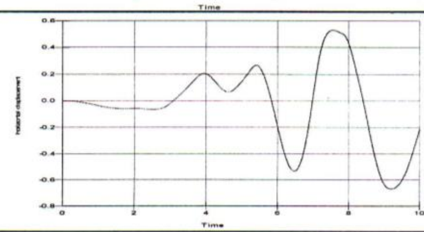
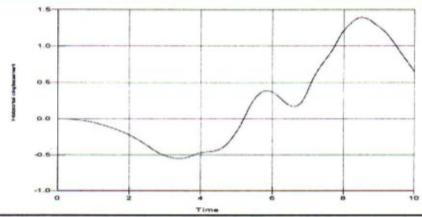
| Liquefaction zones |  |  |
| Node1 Horz. Disp. Versus time |  |  |
| Node2 Horz. Disp. Versus time |  |  |
| Node3 Horz. Disp. Versus time |  |  |
| Node4 Horz. Disp. Versus time |  |  |
| Node5 Horz. Disp. Versus time |  |  |

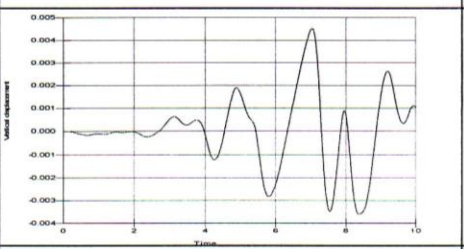
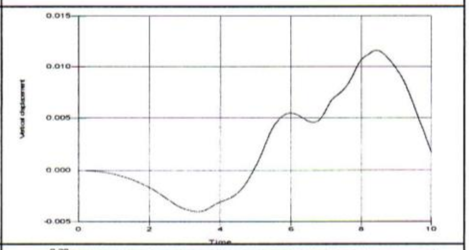
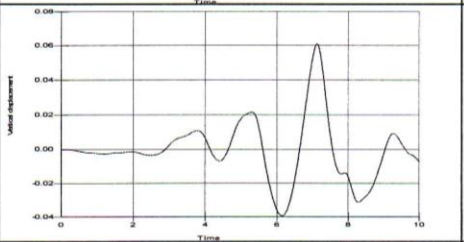
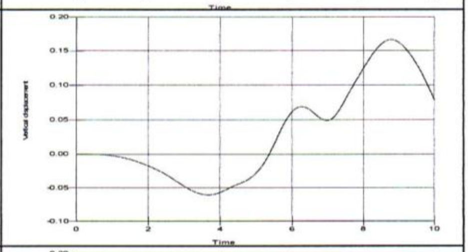
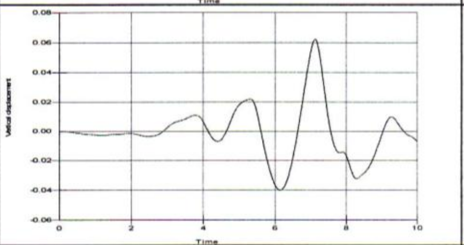
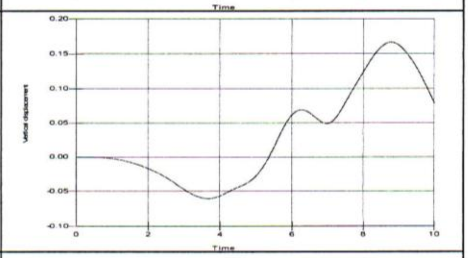
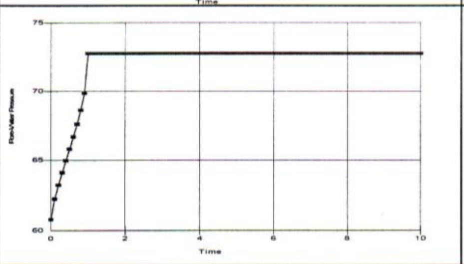
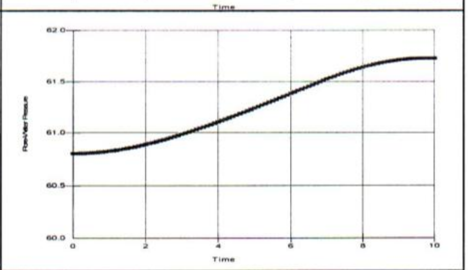
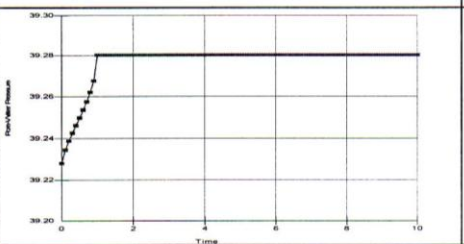
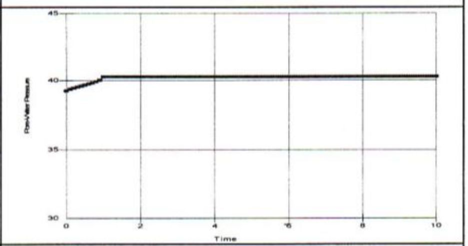
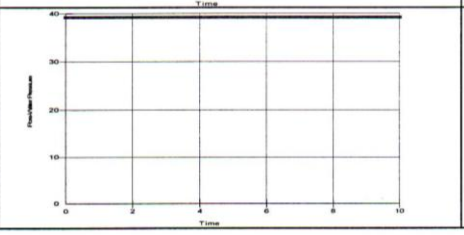
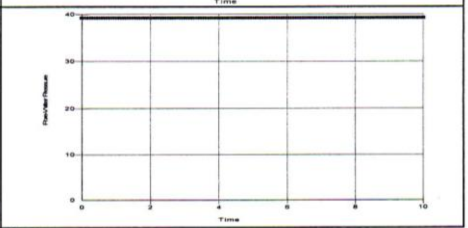
Note: Units: time=Sec., Horizontal displacement= meter.

**Table (5)**

| Case 4 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Liquefa-ction zones |  |  |
| Node1 Horz. Disp. Versus time |  |  |
| Node2 Horz. Disp. Versus time |  |  |
| Node3 Horz. Disp. Versus time |  |  |
| Node4 Horz. Disp. Versus time |  |  |
| Node5 Horz. Disp. Versus time |  |  |

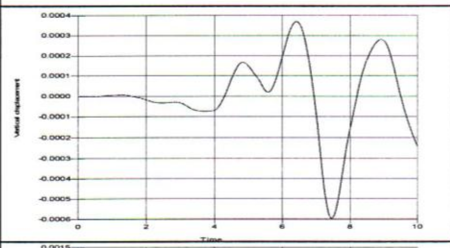
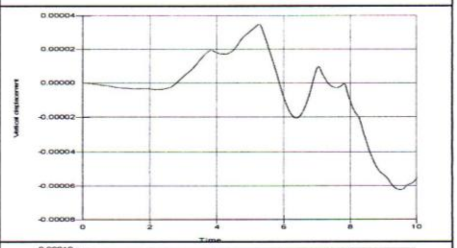
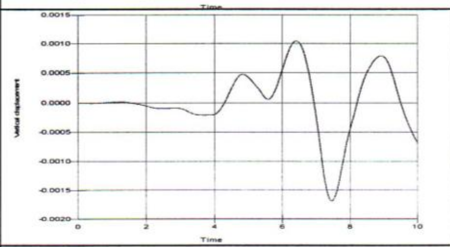
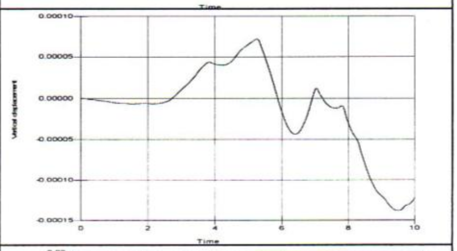
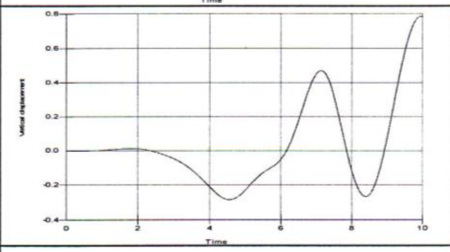
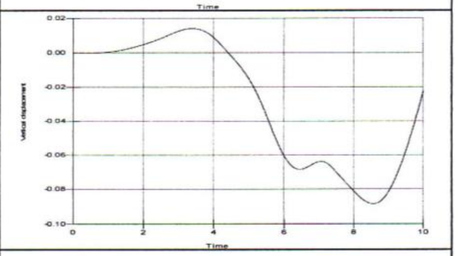
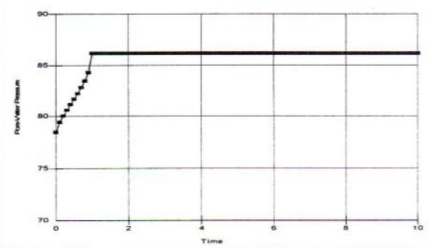
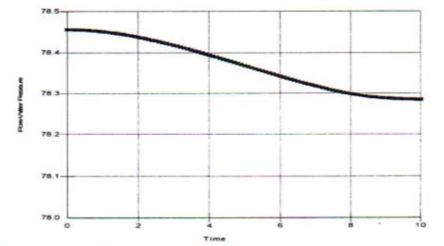
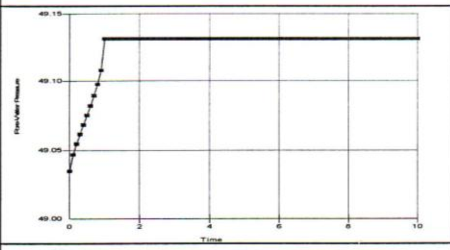
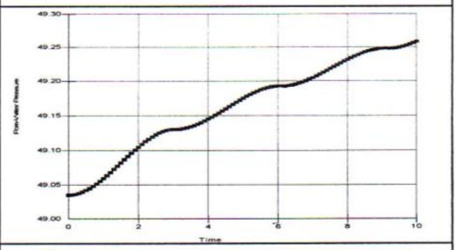
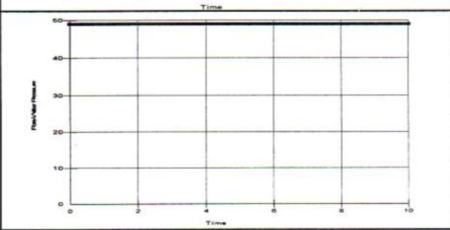
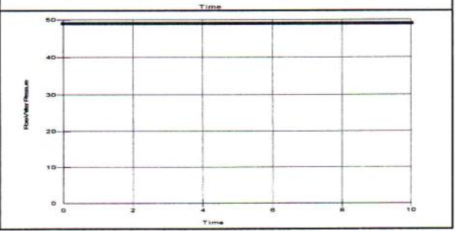
Note: Units: time=Sec., Horizontal displacement= meter.

**Table (6)**

| Case 1 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Node1 Vert. Disp. Versus time |  |  |
| Node3 Vert. Disp. Versus time |  |  |
| Node5 Vert. Disp. Versus time |  |  |
| Node1 PWP Versus time |  |  |
| Node2 PWP Versus time |  |  |
| Node4 PWP Versus time |  |  |

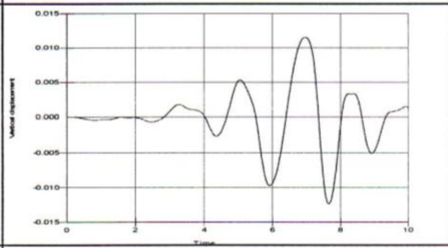
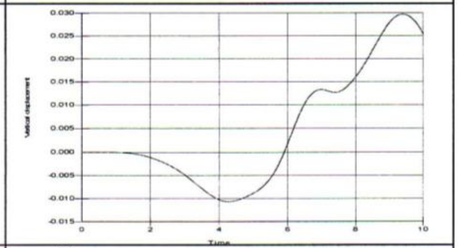
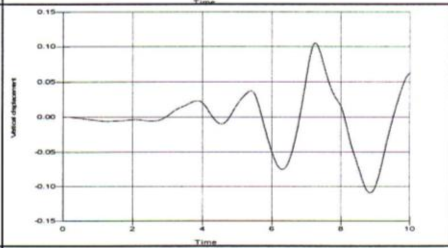
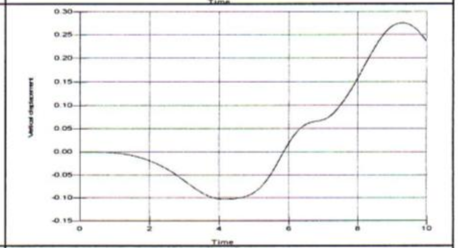
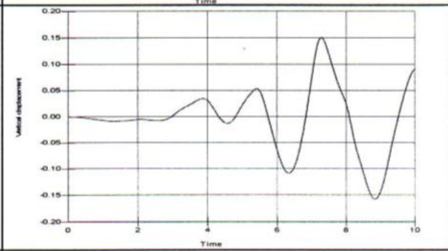
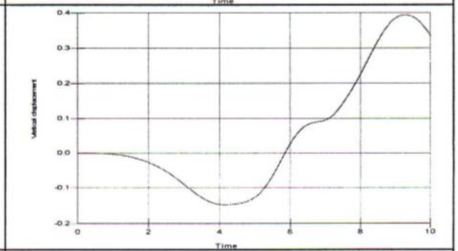
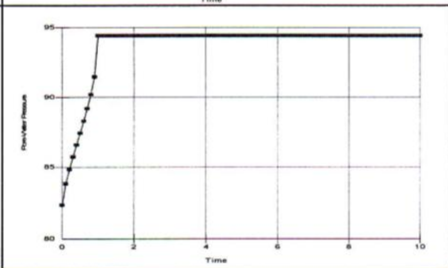
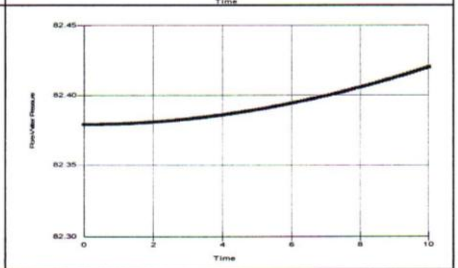
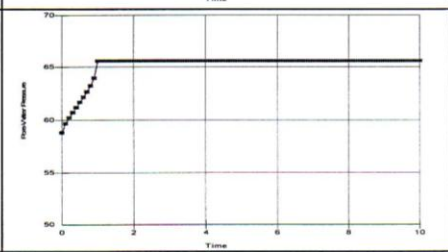
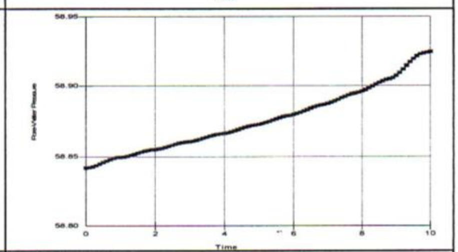
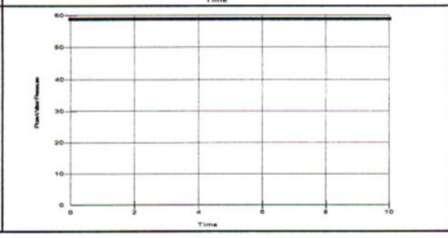
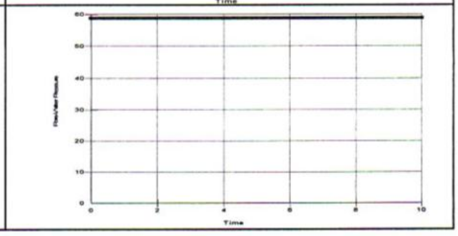
Note: Units: time=Sec., Vertical displacement= meter, Pore water pressure=kPa.

**Table (7)**

| Case 2 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Node1 Vert. Disp. Versus time |  |  |
| Node3 Vert. Disp. Versus time |  |  |
| Node5 Vert. Disp. Versus time |  |  |
| Node1 PWP Versus time |  |  |
| Node2 PWP Versus time |  |  |
| Node4 PWP Versus time |  |  |

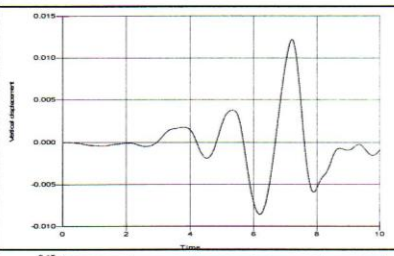
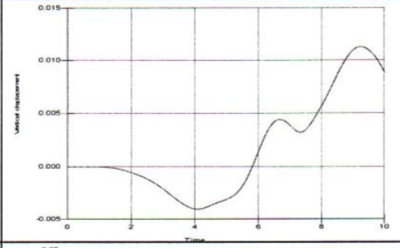
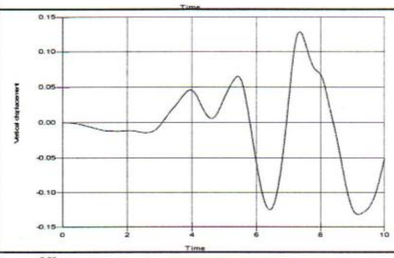
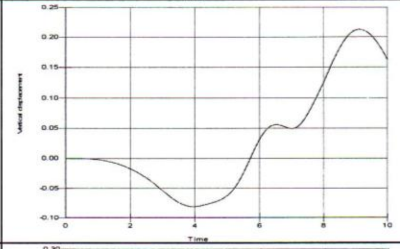
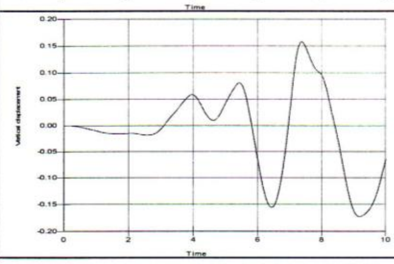
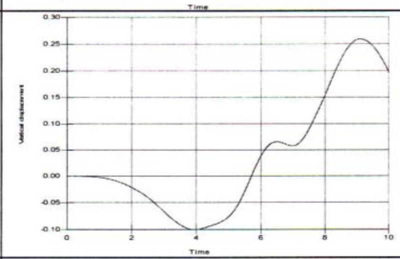
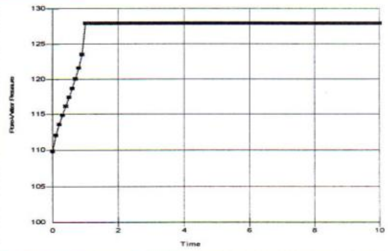
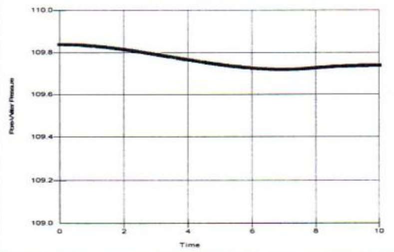
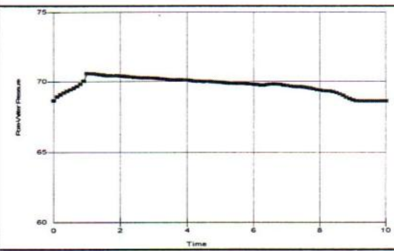
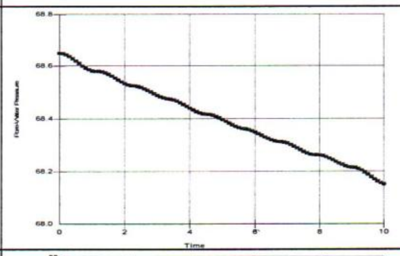
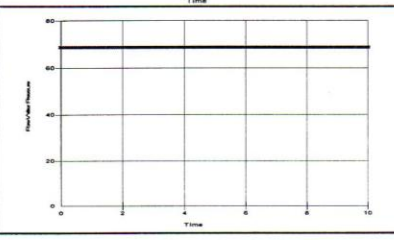
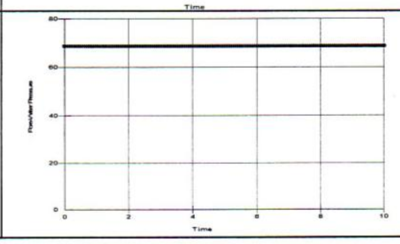
Note: Units: time=Sec., Vertical displacement= meter, Pore water pressure=kPa.

**Table (8)**

| Case 3 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Node1 Vert. Disp. Versus time |  |  |
| Node3 Vert. Disp. Versus time |  |  |
| Node5 Vert. Disp. Versus time |  |  |
| Node1 PWP Versus time |  |  |
| Node2 PWP Versus time |  |  |
| Node4 PWP Versus time |  |  |

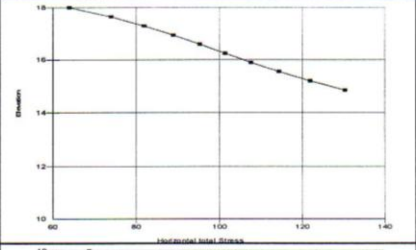
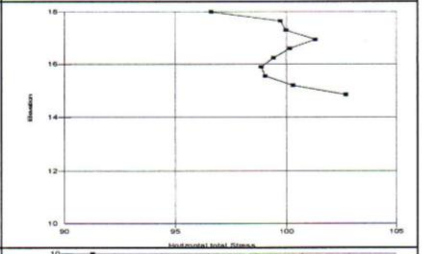
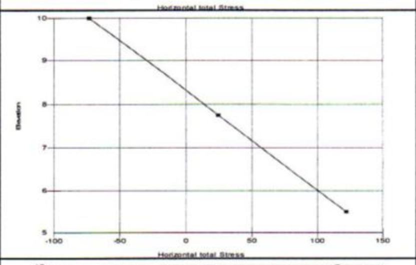
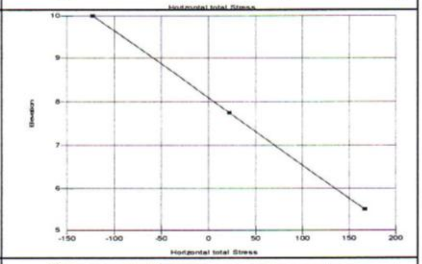
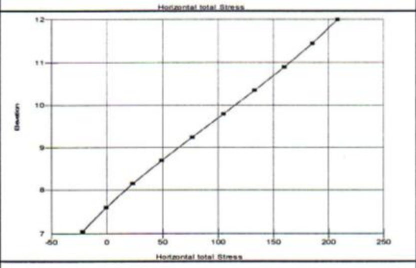
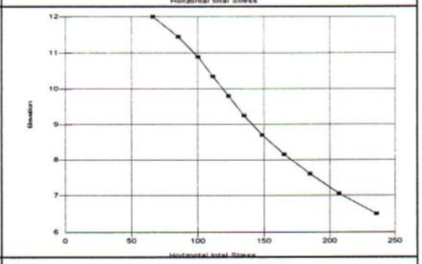
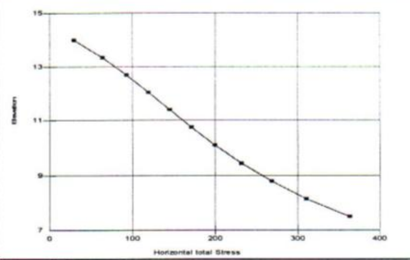
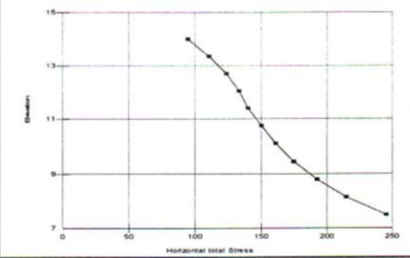
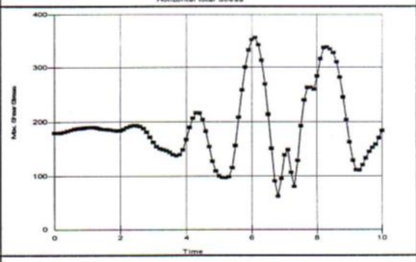
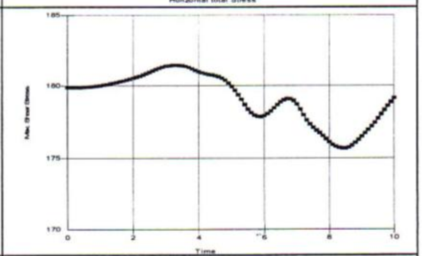
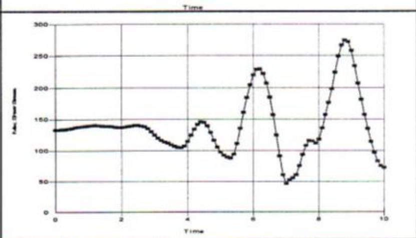
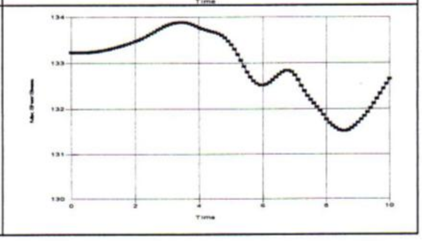
Note: Units: time=Sec., Vertical displacement= meter, Pore water pressure=kPa.

**Table (9)**

| Case 4 | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Node1 Vert. Disp. Versus time |  |  |
| Node3 Vert. Disp. Versus time |  |  |
| Node5 Vert. Disp. Versus time |  |  |
| Node1 PWP Versus time |  |  |
| Node2 PWP Versus time |  |  |
| Node4 PWP Versus time |  |  |

Note: Units: time=Sec., Vertical displacement= meter, Pore water pressure=kPa.

**Table (10)**

| Case No. | Linear Elastic. | Equivalent Linear. |
|---|---|---|
| Case1 Horz. stress Versus Elev. At the Wall face |  |  |
| Case2 Horz. stress Versus Elev. At the Wall face |  |  |
| Case3 Horz. stress Versus Elev. At the Wall face |  |  |
| Case4 Horz. stress Versus Elev. At the Wall face |  |  |
| Case1 Max. shear stress At the Wall Base Versus Time |  |  |
| Case3 Max. shear stress At the Wall Base Versus Time |  |  |

Note: Units: time=Sec., Elevation= meter, Horizontal/Shear stress= kPa.

# (تحليل الجدران الساندة المعرضة إلى قوى زلزالية)

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****جامعة التكنولوجية**

## الخلاصة

تم تحليل الجدران الساندة المعرضة إلى قوى زلزالية باستخدام طريقة عددية ممثلـــة بطريقـــة العناصـــر المحددة ومن خلال تمثيل التربة(رملية مشبعة) بحالتين: الحالة المرنة، الحالة الخطية المكافئة في حين تم تمثيل الجدار بالحالة المرنة فقط. وتم دراسة الإزاحة الأفقية والعمودية وضغط ماء المســـام المتولـــد خـــلال الهـــزة الأرضية والإجهاد الأفقي الكلي في التربة بالقرب من الجدار والإجهاد القصي الأعلى بالتربة عند قاعدة الجدار وبإبعاد مختلفة للجدار. وقد تبين أن الحالة الخطية المكافئة للتربة بالمقارنة مع الحالة الخطية تعطي نتـــائج أدق ولوحظ تمركز مناطق الفوران (Liquefaction) في جانب التربة غير الفعالة في حين وصلت الإزاحة الأفقية القصوى عند قمة الجدار إلى(0.67) متر، وزادت الإزاحة العمودية بحدود%(116-66) مع ازدياد أبعاد الجدار. وازدادت كل من ضغط ماء المسام/الإجهاد الكلي الأفقي مع الزمن/تغيـــر الأبعـــاد بحـــدود(37%)،(200%) بالتتابع.