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Earthfill Dams Response to Earthquake Excitation -Khassa Chai Dam as a Case Study

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Abstract

In this paper, a dynamic analysis has been carried out on zoned earthdam subjected to earthquake excitation in which pore water pressure, effective stresses and displacements are calculated. The finite element method is used and the computer program Geo-Studio is adopted in the analysis through its sub-programs SEEP/W and QUAKE/W. A case study is considered to be Khassa Chai dam which is located on Khassa Chai river north of Iraq and consists of zoned embankment with a total length of 3.34 km. The selected earthquake for the analysis is El-Centro earthquake with a period of 10 sec and different amplitudes of acceleration. The time of the analysis is taken as 600 sec. with a time step (Δt = 0.05 sec.) to investigate the behavior of the soil for a period of time after the earthquake has stopped, a free vibration period is included in the analysis. It was concluded that the value of pore water pressure generated at the base of the core is greater than that in the upper parts of dam. The horizontal and vertical effective stresses continue to decrease during the period of analysis 600 sec. which indicates that the soil continues to weaken during this period, the horizontal displacement increases with depth of the point from the crest and the largest horizontal displacement will be at the base of the dam at time 60 sec and There is attenuation of the acceleration to some degree depending on the amplitude of the input horizontal acceleration. The maximum horizontal displacement decreases by about 37%, 45% and 49% when using a horizontal drain 2 m thick at the downstream under a peak acceleration of 0.05g, 0.1g and 0.2g, respectively.

Keywords: Earthfill Dam, Dynamic, Response, Earthquake, Liquefaction.

Introduction:

Earth dams have been built since early times to store or divert water for irrigation. Embankment dams and other critical geotechnical structures require consideration of the effects of earthquake. Numerous embankment dam failures have been attributed to seismic loading and this is a critical case for embankment design. Since the failure of a dam is far more disastrous to community than that of other structure. Earthquakes can affect embankment dams by causing settlement, instability, internal cracking, differential movements or damage to associated structures.

For a seismic design of dam, the aim of designer is to know the force exerted on the dam structure due to the probable ground motion expected at the site, as well as to estimate the behavior of a dam during an earthquake.

Seed et al. (1975) [9] studied the dynamic analysis of Lower San Fernardo dam which was subjected to San Fernardo earthquake in 1971 with maximum acceleration of 0.55g and with a period of about 15 sec. The finite element method was used to represent the cross section of the dam. By using average consolidated undrained strength parameters, the factor of safety along the critical sliding is about 1.06. Using the constructive value (after the failure of the dam) of strength parameters, the factor of safety is about 0.8. The stability of the embankment was evaluated by investigating the liquefaction and strain potential within many potential zones of failure. It was found that the peak accelerations (and therefore the peak stresses) occurred at approximately 10 sec. after the start of the motion.

Daghigh (1993) [2] studied the behavior of Alavian earthdam in Iran when subjected to an earthquake using EL-Centro with maximum acceleration of 0.35g through the program DIANA. It was concluded that the constrained and free condition of the vertical boundaries had effect on the computed response of the dam. The stiffness of the tuff rock and the level of the base rock at which the input motion was applied in the analysis, can affect the computed response of the dam. The damping had dramatic effect on the amount of the computed response of the dam, the smaller the damping ratio, the larger the computed response. The computed response at the top of the dam was higher than at the other parts. The homogeneous dam behaves stiffer than the heterogeneous one.

Gui et al. (2006) [6] studied the dynamic behavior of Renyi-Tan earth dam in Taiwan during the 921-jiji earthquake. Dynamic analysis was carried out using the program FLAC. The dynamic behavior of the dam was evaluated in terms of dam displacement, excess pore water pressure, and acceleration recorded throughout the depth of the dam. From the results of the dynamic analysis, it was found that when water table is 12 m, the upstream slope generates a 4 cm displacement, and the downstream slope generates a 5 cm displacement. When water table is 24 m, the upstream slope generates a 2 cm displacement, and the downstream slope generates a 5 cm displacement and that the maximum displacement contour is generated between t = 20 and 30 sec, which coincides with the main earthquake strike. After the main strike, no further displacement was seen generated, excess pore water pressure would be generated in saturated zones of the dam.

Moayed and Ramzanpour (2008) [8] studied the dynamic behavior of a zoned core earth-fill dam using two dimensional finite element analysis program Geo- Slope. Mamloo dam was designed as zoned core that is composed of three vertical zones including central lean clay core and two sides clayey gravel layers. The embankment dam was analyzed for the maximum operating reservoir level. Behavior of dam was analyzed in two cases including homogenous clayey core and zoned core. The study showed that the maximum horizontal acceleration at the zoned core dam crest reached 0.0206g and in simple core reached 0.0207g. The maximum x-acceleration responses of cutoff were 0.00873 m^2/s and 0.00842 m^2/s in zoned core and simple core. Maximum horizontal displacement response of zoned core dam crest was equal to 4.15 cm but in simple core it was 4.3 cm, and the horizontal displacement response of cutoff in both cases was 1.4 cm.

Khattab and Khalil (2013) [7] carried out dynamic analysis within the clay core of Al-Mosul earth dam considering the saturated/unsaturated conditions using Geo-Slope software. Three selected sections through the dam were chosen for the analysis (in the middle, right, and left sides of the dam to cover the dam body). The investigation of the dam body response to the earthquakes with many values of the maximum horizontal acceleration was done under normal, maximum, and minimum operation water levels. Transient and steady state analysis of pore water pressure was performed. It was shown that the maximum pore water pressure occurred in the nodes in the upstream near of the core base at the time during and after the end of earthquake shaking. The results of the study also presented a positive pore water pressure development in the lower part of the core when the water was at maximum, normal, and minimum operation levels, with negative values near the crest of the dam.

The aim of this study is to carry out a dynamic analysis of earthfill dams under the effect of earthquakes. As a case study, Khassa Chai dam is selected. The study includes investigating the effect of some parameters such as the maximum acceleration, horizontal motion, the geometric design of dam, and properties of soils in different zones of the dam. In addition, a trial shall be made to improve the dam performance under the effect of earthquakes using the program QUAKE/W.

Description of Khassa Chai Dam

Khassa Chai Dam is located on Khassa Chai River upstream Kirkuk City. The Khassa Chai River is a tributary of Zaghitun River which is flowing into the existing Al-Adhaim Dam Reservoir. The dam site is located near Kuchuk Village, 10 km northeast of Kirkuk City north of Baghdad in Iraq. The dam consists of composite section of pervious and impervious materials. The shell of the dam consists of pervious material, and a core of impervious materials.

Embankment zoning provides an adequate impervious zone, transition zones between the core and the shells, seepage control, and stability. The base of the dam is composed of a series of formations, which vary according to the depth from stiff to very stiff brown silty clay with a lot of medium to coarse grained gravel and black traces of organic matter. The shell consists of sand and gravel, the upstream slope is (1:3 vertical: horizontal) while at the downstream is (1:2.5), and the top width is 14 m. The central core is composed of silty clay with a slope of 1:1 for both upstream and downstream sides, the core top width is 13 m and this width gradually increases until it reaches 128 m at the base. Chimney drainage is adopted downstream of the core with 2 m thickness as illustrated in Figure 1 (Center of Designs and Studies, 2007 [1]).

Seepage Analysis of Khassa Chai Dam

Seepage through and under the dam is analyzed using the program SEEP/W. The upstream boundary nodes are designated as head boundaries with total head equals to the water level in the reservoir, and the downstream boundary nodes are designated with total flux equals to zero. The bottom nodes along the foundation are designated as a zero discharge (no flow).

Initial static stress analysis of Khassa Chai dam

Prior to the dynamic analysis, it is essential to establish initial static stress conditions. The initial static stress of Khassa Chai dam subjected to the force of gravity was computed by a separate step within the QUAKE/W computer program. The computed initial static stress results were in very good agreement. Then, the computed results of the initial static stress and initial pore water pressure were then imported to the dynamic analysis part of the QUAKE/W program.

Dynamic Analysis

After the seepage analysis is done, the dam is analyzed by the program QUAKE/W depending on results obtained from the program SEEP/W. The finite element mesh used for the analysis is shown in Figure 2. The mesh includes higher-order six-noded triangular elements. In dynamic analysis, the left and right vertical boundary conditions on nodes are assumed to be free to move in the horizontal direction but they are fixed in the vertical direction (Fattah and Nsaif, 2012 [4] and Fattah et al., 2015 [5]). The boundary conditions along the horizontal base of the foundation are assumed to be restrained in the vertical directions and free in the horizontal direction. Linear elastic model is used in the analysis. For this model, the shear modulus (G) is calculated for the soil layers of the dam and its foundation. The damping ratio is assumed to be 0.02. The material properties of different zones in the dam body and its foundation are listed in Table 1, from the report prepared by the Center of Designs and Studies, (2007) [1]. The dam is assumed to be subjected to earthquake excitation of El-Centro earthquake which occurred in 1941. The acceleration-time history of this earthquake is shown in Figure 3. This earthquake has been selected due its reliable data which have been filtered and it is conventionally used in dynamic analyses.

Effect of Earthquake on Khassa Chai Dam

Only the horizontal component of motion is considered in the analysis by studying its effect on Khassa Chai dam with a horizontal drain 2 m thick. The peak acceleration of the input horizontal earthquake record is modified to three values; 0.05 g, 0.1 g and 0.2 g. So that the earthquake record is scaled for each case. For the previous cases, three water levels are taken as:

- Minimum water level 26 m.
- Normal water level 40 m.
- Maximum water level 55 m.

The results are shown in the form of figures which include: pore water pressure, vertical and horizontal effective stress, x-displacement and x-acceleration -time history for different nodes selected to represent different locations through the dam such as shell, core and foundation., displacements along two sections through the dam body at time 60 sec., liquefaction zones and pore water pressure contours at time 600 sec. These results will be discussed in the following sections. A horizontal drain 2 m thick is assumed to be constructed in the dam downstream. The drain is assumed to have a coefficient of permeability of 0.01 m/s, other material properties are listed in Table 1. Figures 4 and 5 show the results of nodes 3 and 5 under a maximum horizontal acceleration of 0.05 g, similar results were obtained for a maximum horizontal acceleration of 0.1 g and 0.2 g and minimum water level. Figures 6 and 7 show the results of nodes 3 and 5 under a maximum horizontal acceleration of 0.05 g, similar results were obtained for a maximum horizontal acceleration of 0.1 g and 0.2 g and normal water level. Figures 8 and 9 show the results of nodes 3 and 5 under the same maximum horizontal ground acceleration and maximum water level. Figure 11 shows the response of node 1 for the case of maximum water level. Figure 24 shows the displacement along two sections through the dam for maximum water level at time 60 sec. Figure 12 shows liquefaction zones through the dam section at the end of analysis, time 600 sec.

Minimum water level

The results revealed that the value of pore water pressure at node 3 increases gradually until it reaches 113 kPa, then leveled off where the liquefaction begins at almost the same time 560 sec. for input accelerations of 0.05 g, 0.1 g and 0.2 g. Figures 6, 8 and 10 show the result of node 5.

Normal water level

A comparison between the results shows that the maximum value for pore water pressure at node 3 is 195 kPa and the time of initiation of liquefaction is decreased as the input acceleration increased until it reaches (507 sec) at input acceleration of 0.2 g, while at node 5, the liquefaction takes place at the early times of the analysis at time 59 sec. and the value of the maximum pore water pressure is 89 kPa as shown in Figures 12, 14 and 16.

Maximum water level

A comparison between the results shows that the response of node 3 is almost the same when the input acceleration is 0.05g, 0.1g and 0.2

g where liquefaction starts at earlier time 350 sec. The response of node 5 which reveal that as the input acceleration increased, the time for liquefaction to take place is decreased, where it begins at earlier time 277 sec. when the input acceleration is 0.05g and decreased to 213 sec. for the input acceleration of 0.2g. The horizontal and vertical effective stresses continue to decrease during the period of analysis and the results are very close to the results for the case of the dam section with filters only.

Figure 10 shows the response of node 1, located at the dam crest and the results are summarized in Table 2. It can be concluded that there is attenuation of the acceleration to some degree depending on the amplitude of the input horizontal acceleration. The attenuation ranged between 16-44%. The maximum horizontal displacement decreases by about 37%, 45% and 49% when using a horizontal drain 2 m thick at the downstream.

As the water level increases from minimum to maximum and as the input acceleration increases from 0.05g to 0.2g, it can be noticed that the liquefaction starts to take place after the earthquake has stopped in the upstream shell at the minimum water level, liquefaction zone will continue to grow and raise as the water level raised until it reaches the core as shown in Figure 12. These results are compatible with those obtained by Fattah and Nsaif (2010) [3].

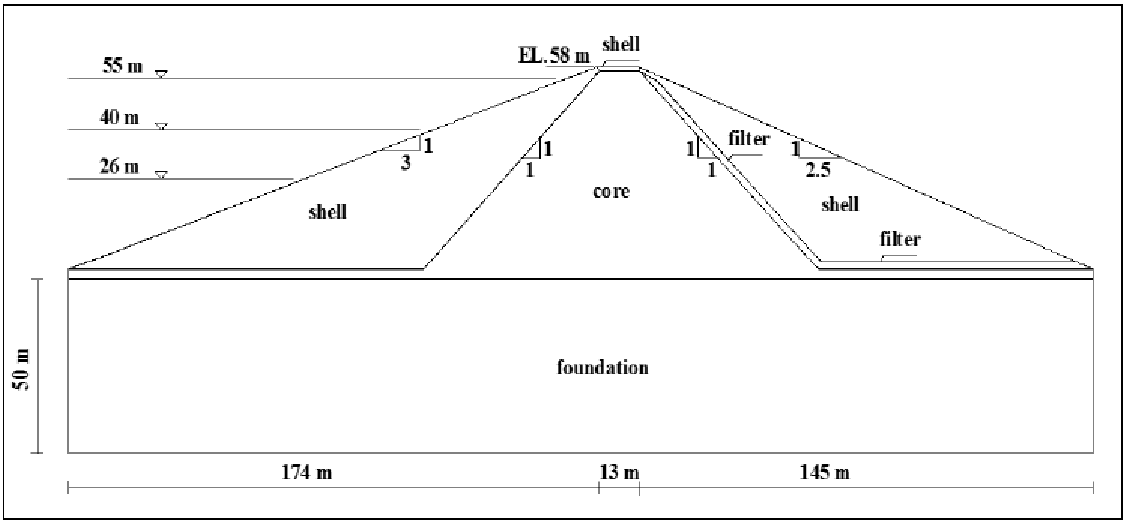


Figure 1: Typical cross-section of the dam (after the Center of Designs and Studies, 2007 [1]).

Table 1: Material properties of Khassa Chai dam (Center of Designs and Studies, 2007 [1]).

Material	Dynamic elastic modulas E (kN/m^2)	Poisson's ratio (ν)	Unit weight (kN/m^3)	Coefficient horizontal of permeability, k_h (m/s)	Volumetric water content (%)
Shell	19000	0.3	18	$1.25x10^{-2}$	15
Core	30000	0.45	20	$2.25x10^{-10}$	25
Filter	19000	0.3	18	$1.25x10^{-5}$	15
Foundation	15000	0.48	19	$1x10^{-10}$	30
Drain	150000	0.28	23	$1x10^{-2}$	10
Blanket	30000	0.45	20	$2.25x10^{-10}$	25

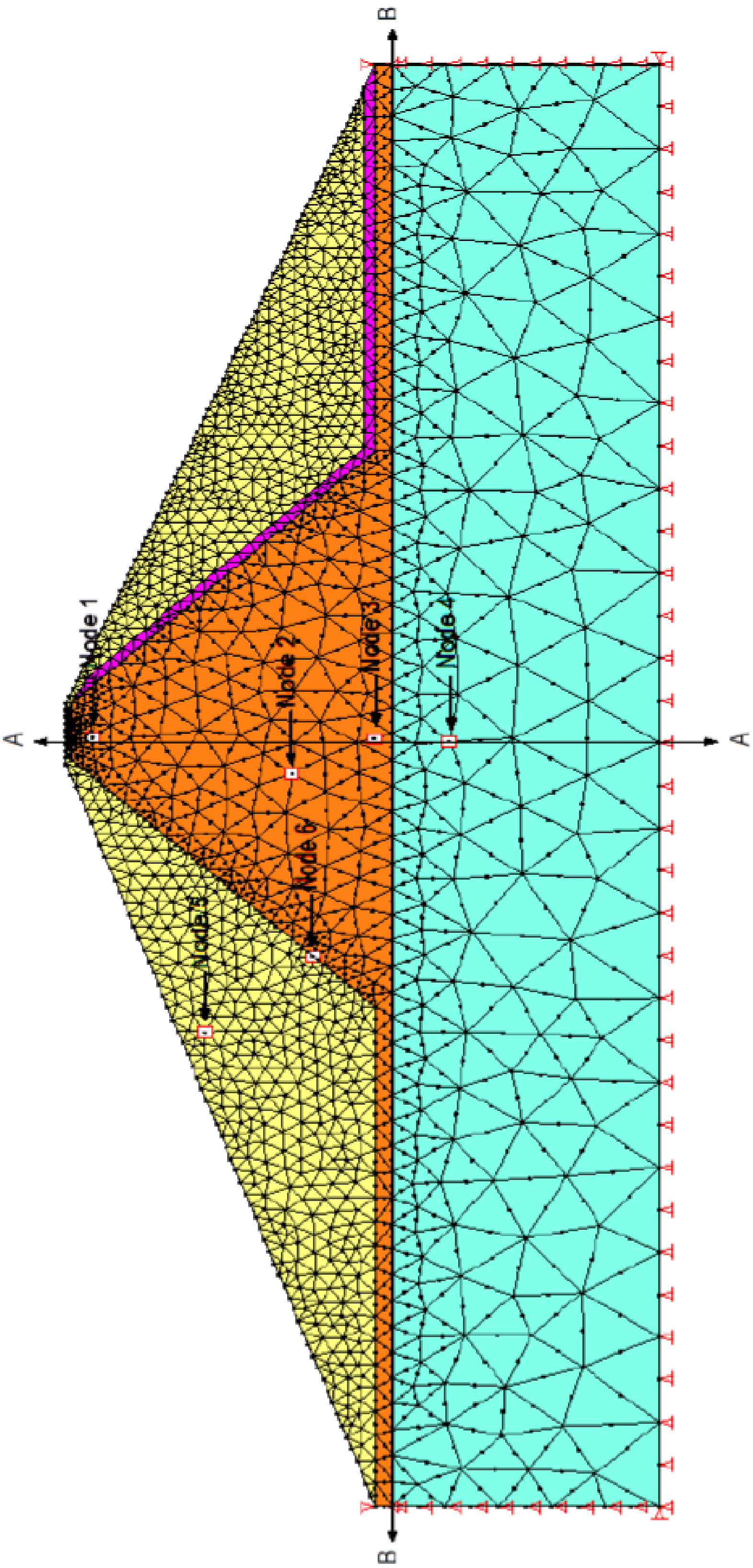


Figure 2: Finite element mesh for dynamic analysis of Khassa Chai Dam.

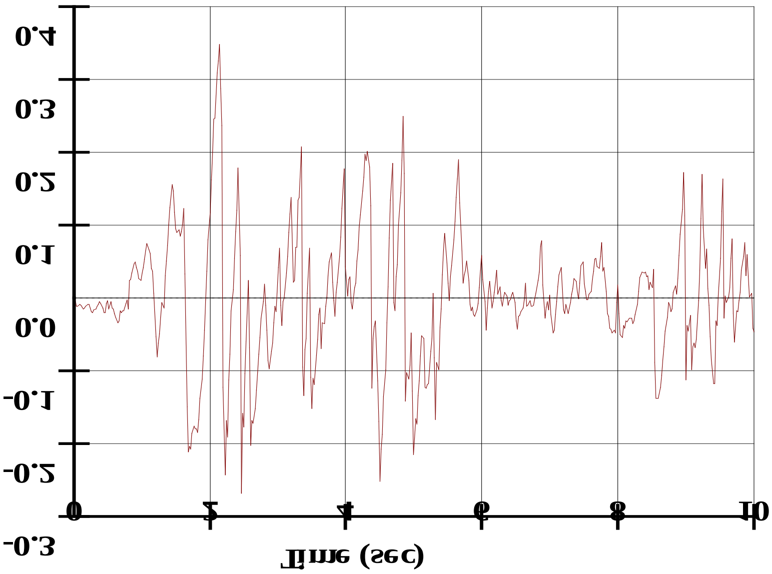


Figure 3: The Acceleration-time history record for El-Centro earthquake, (QUAKE/W, 2004).

Table 2: Acceleration response of node 1 under maximum water level with the presence of horizontal drain 2m thick.

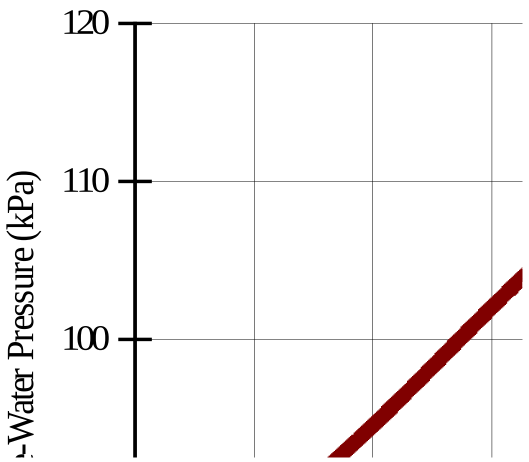
Acceleration (g)	Maximum X-displacement (m)	Maximum X-acceleration (m/sec^2)	Maximum X-acceleration (g)
0.05	0.0521	0.413	0.042
0.1	0.0625	0.548	0.056
0.2	0.0912	1.18	0.12

Table 3 summarizes the minimum and maximum values of the deviatoric stress at different water levels. The value of maximum deviatoric stress increased in this case by about 63% compared with the case of the dam section with filters only.

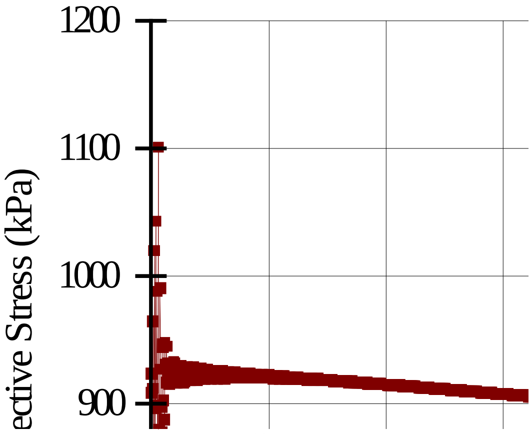
Table 4 shows the maximum values of pore water pressure and the time of their occurrence under different water levels and ground acceleration of 0.2g. The pore water pressure decreases with the presence of horizontal drain by about 2.4-3.8%. Gravel materials of the drain causes to dissipate pore pressure that is reproduced from rearrangement of particles under earthquake loading. So particles of core zone in zoned core dam bear less pore pressure and more effective stress than in simple core dam because there is not much water flowing in this area, there is only a small fraction of excess pore water pressure is generated. In the upstream shell, which is mainly gravel, excess pore pressure is least generated.

Table 3: Computed values of deviatoric stress through the dam with a horizontal drain 2 m thick under a maximum horizontal acceleration of 0.2g.

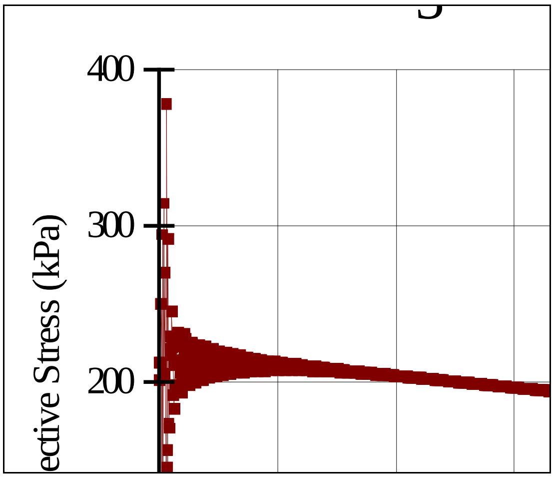
Water level	Deviatoric stress (kPa)	
	Minimum	Maximum
Minimum	5.27	1854.35
Normal	6.76	1850.42
Maximum	4.84	1972.47



(a) Pore water pressure

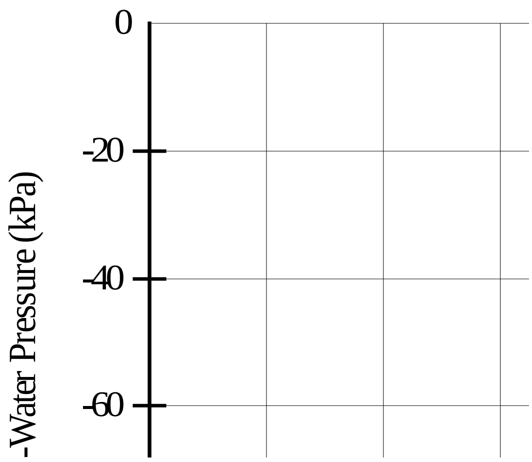


(b) Vertical effective stress

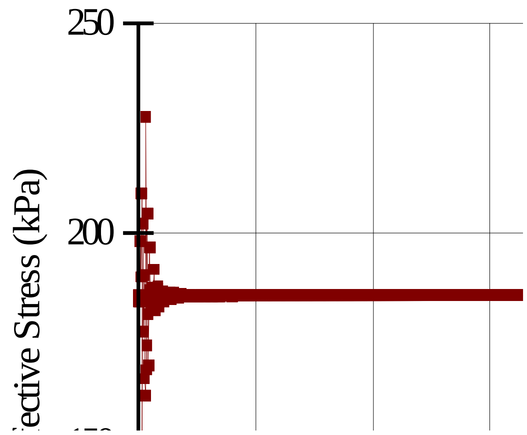


(c) Horizontal effective stress

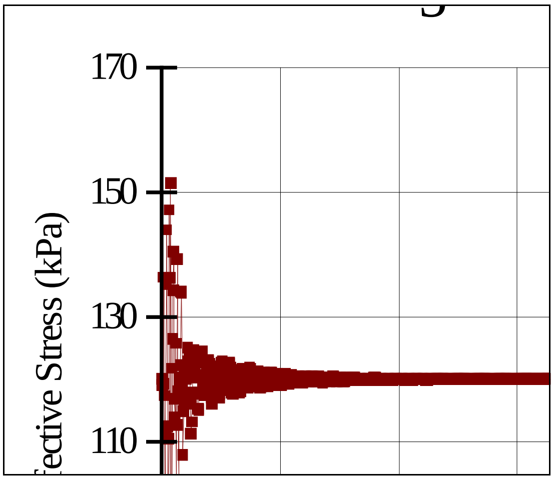
Figure 4: Earthquake response of node (3) under a maximum horizontal acceleration of 0.05 g, with a horizontal drain 2 m thick, minimum water level.



(a) Pore water pressure



(b) Vertical effective stress



(c) Horizontal effective stress

Figure 5: Earthquake response of node (5) under a maximum horizontal acceleration of 0.05 g, with a horizontal drain 2 m thick, minimum water level.

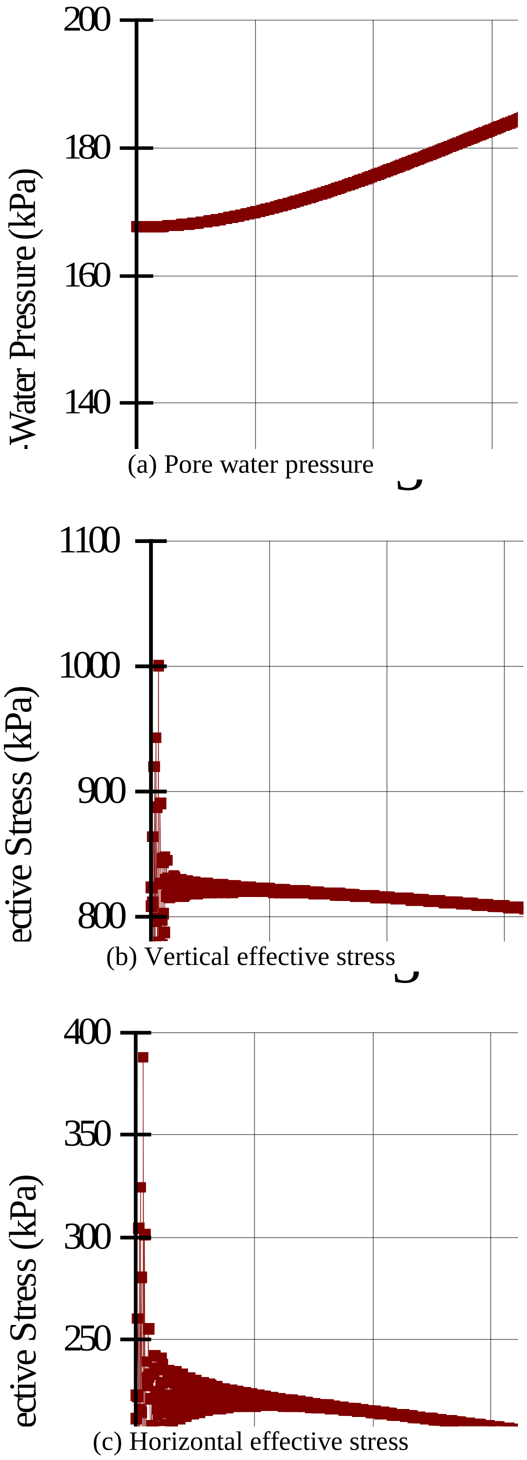


Figure 6: Earthquake response of node (3) under a maximum horizontal acceleration of 0.05 g, with a horizontal drain 2 m thick, normal water level.

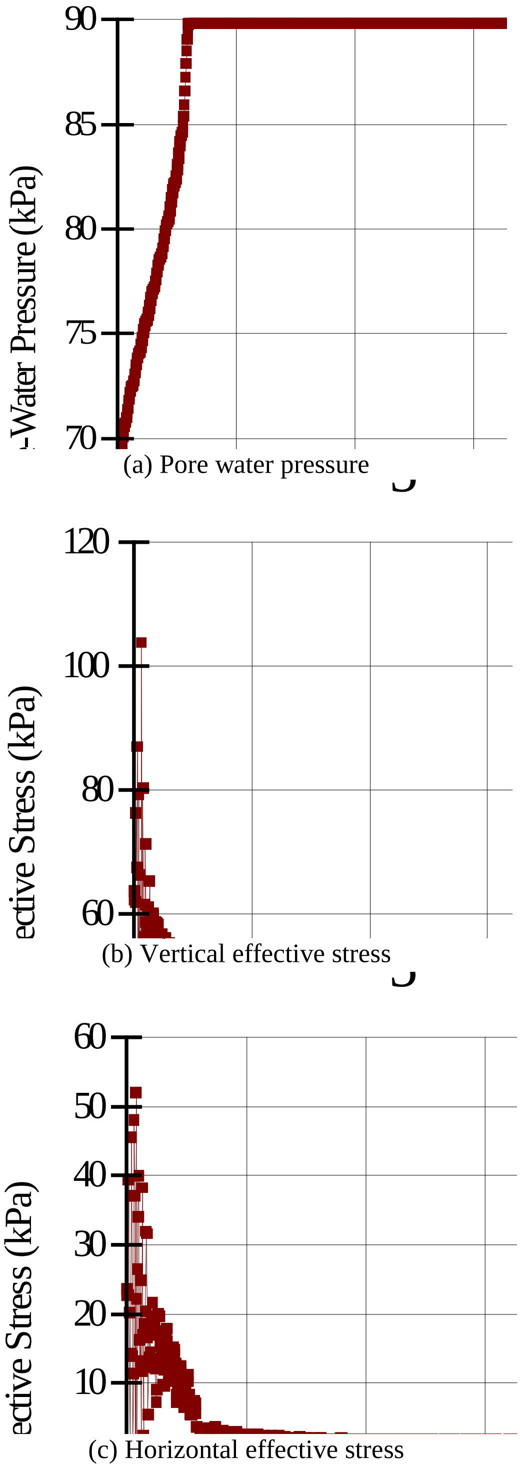
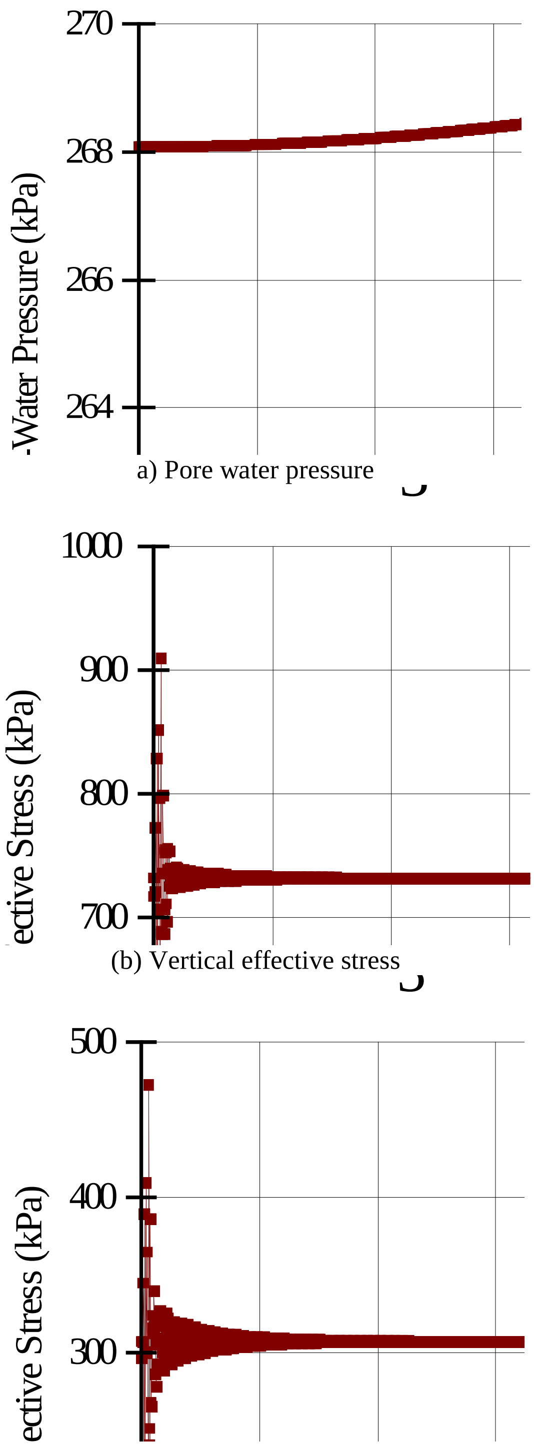


Figure 7: Earthquake response of node (5) under a maximum horizontal acceleration of 0.05 g, with a horizontal drain 2 m thick, normal water level.

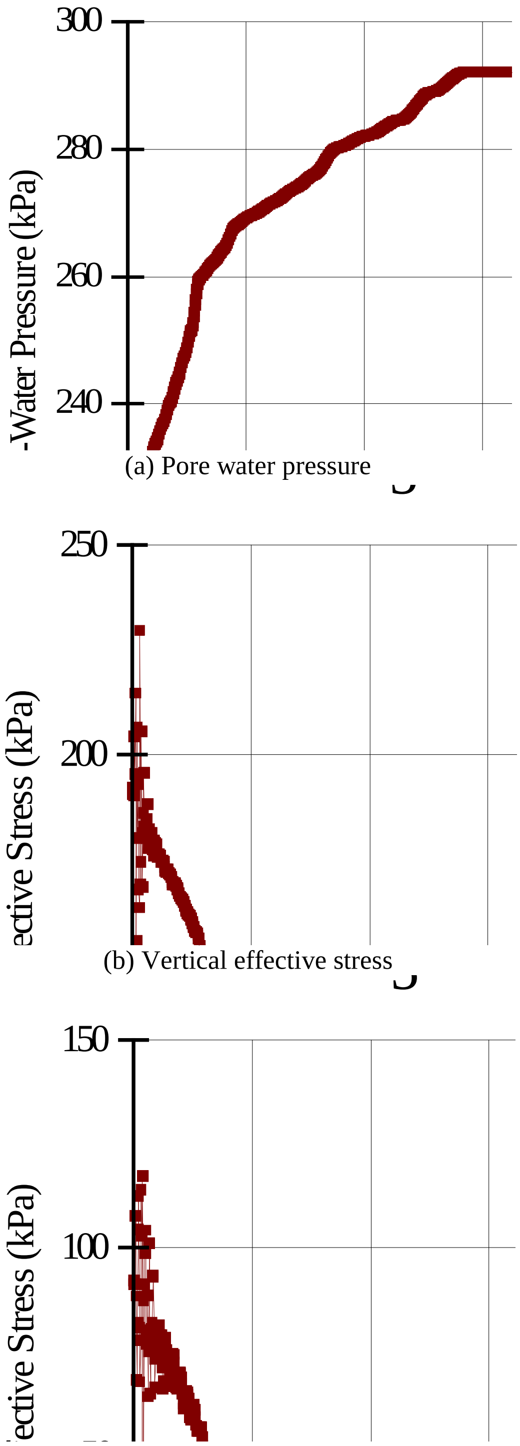


a) Pore water pressure

(b) Vertical effective stress

(c) Horizontal effective stress

Figure 8: Earthquake response of node (3) under a maximum horizontal acceleration of 0.05 g, with a horizontal drain 2 m thick, maximum water level.

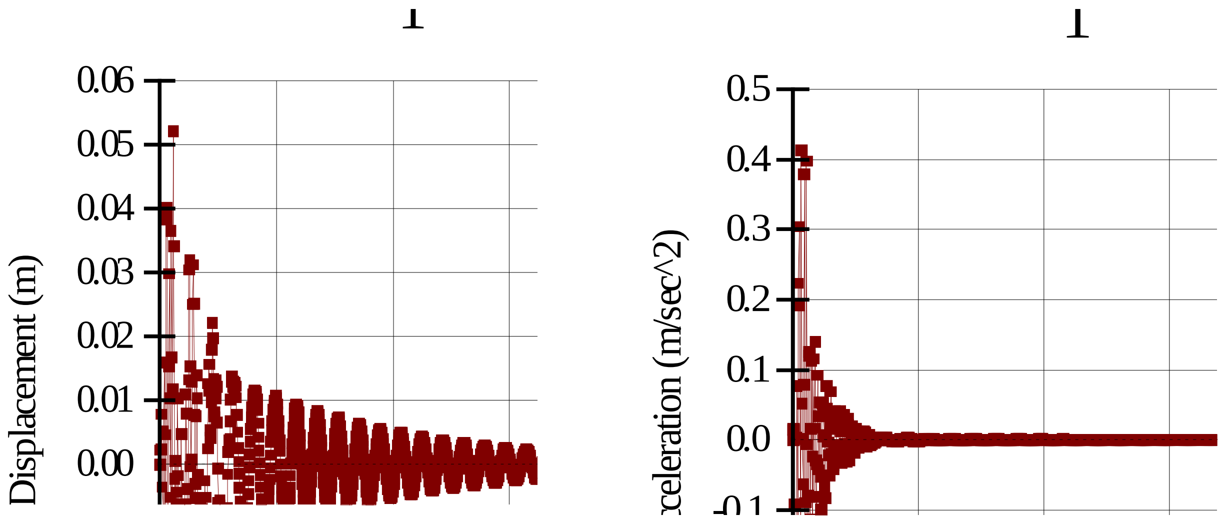


(a) Pore water pressure

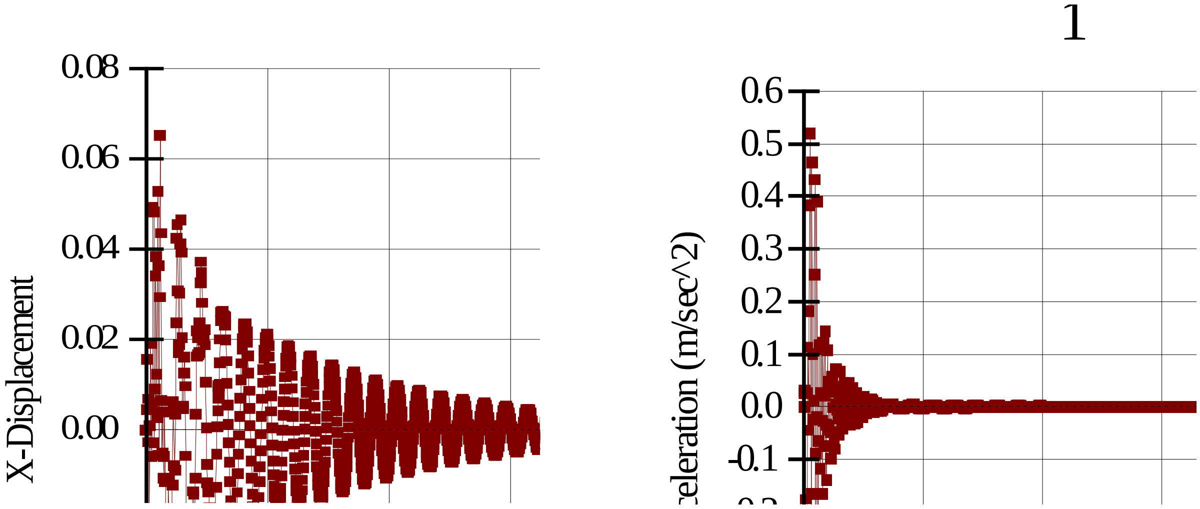
(b) Vertical effective stress

(c) Horizontal effective stress

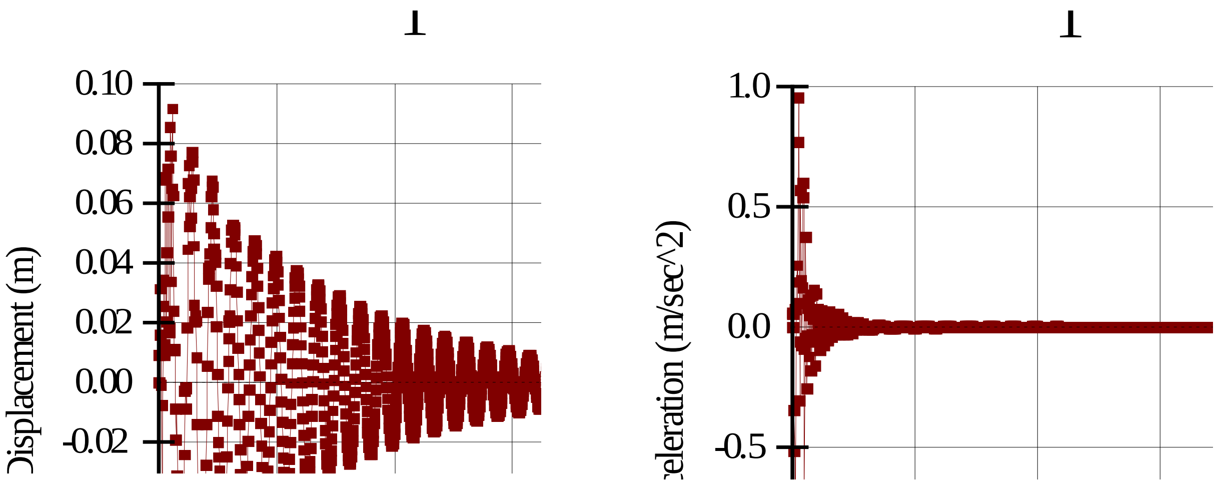
Figure 9: Earthquake response of node (5) under a maximum horizontal acceleration of 0.05 g, with a horizontal drain 2 m thick, maximum water level.



(a)Horizontal displacement at 0.05g.

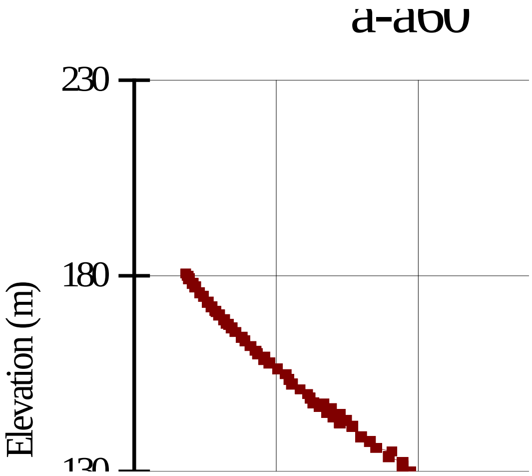


(b) Horizontal displacement at 0.1g.

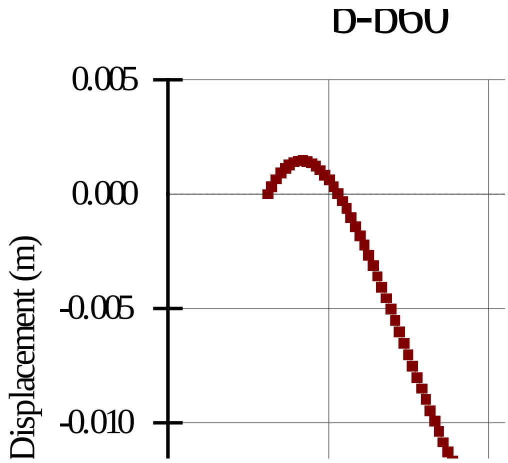


(c) Horizontal acceleration at 0.2 g

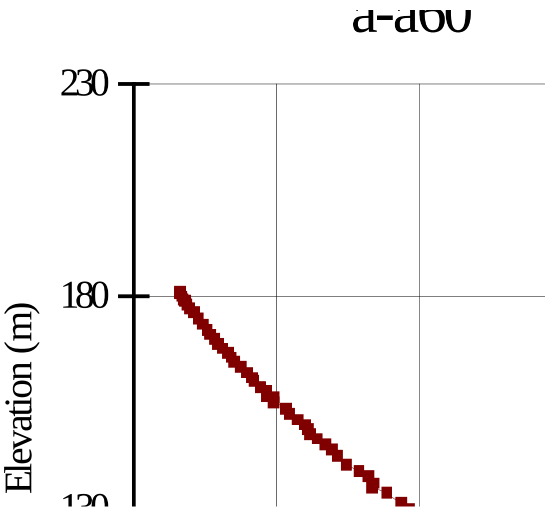
Figure 10: Earthquake response of node (1) at maximum water level, with a horizontal drain 2 m thick.



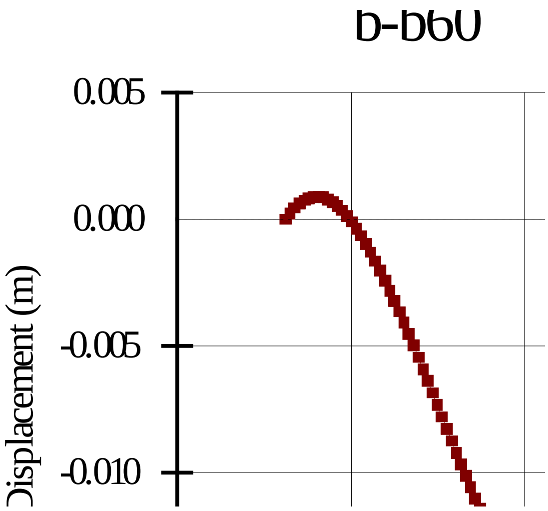
(a) X- Displacement sec.(A-A) at 0.05 g.



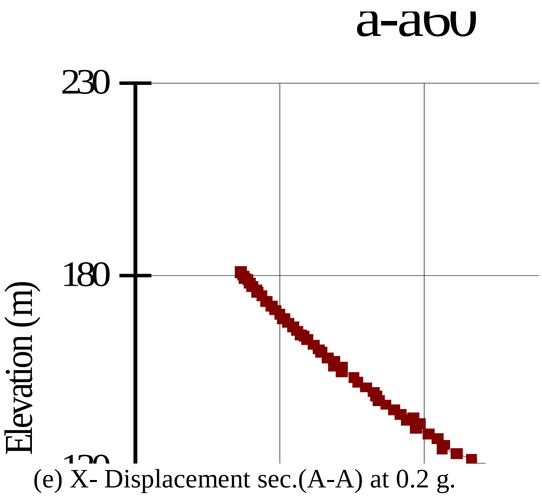
(b) Y-Displacement sec.(B-B) at 0.05 g.



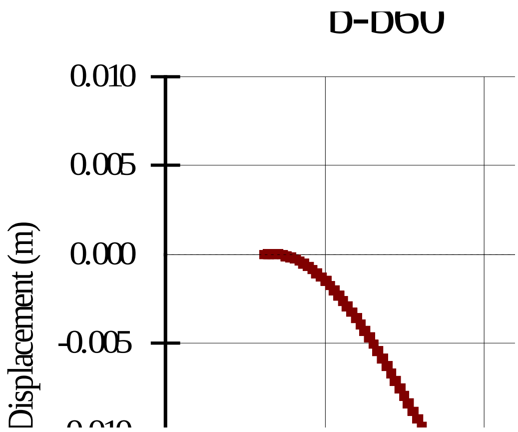
(c) X- Displacement sec.(A-A) at 0.1 g



(d) Y-Displacement sec.(B-B) at 0.1 g

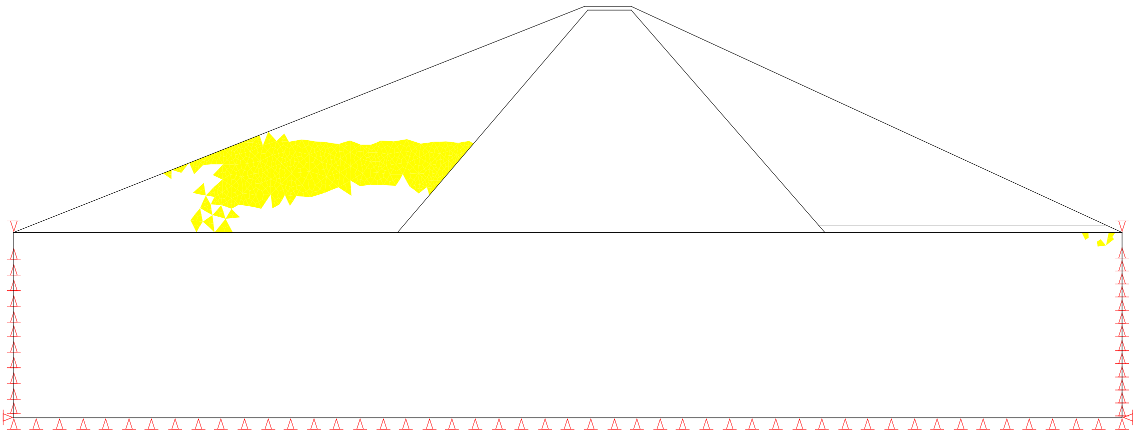


(e) X- Displacement sec.(A-A) at 0.2 g.

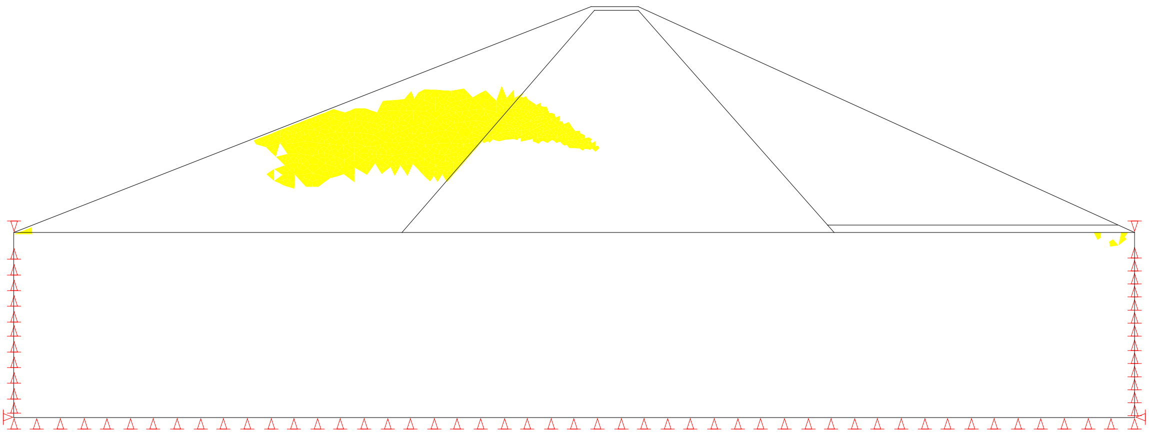


(f) Y-Displacement sec.(B-B) at 0.2 g.

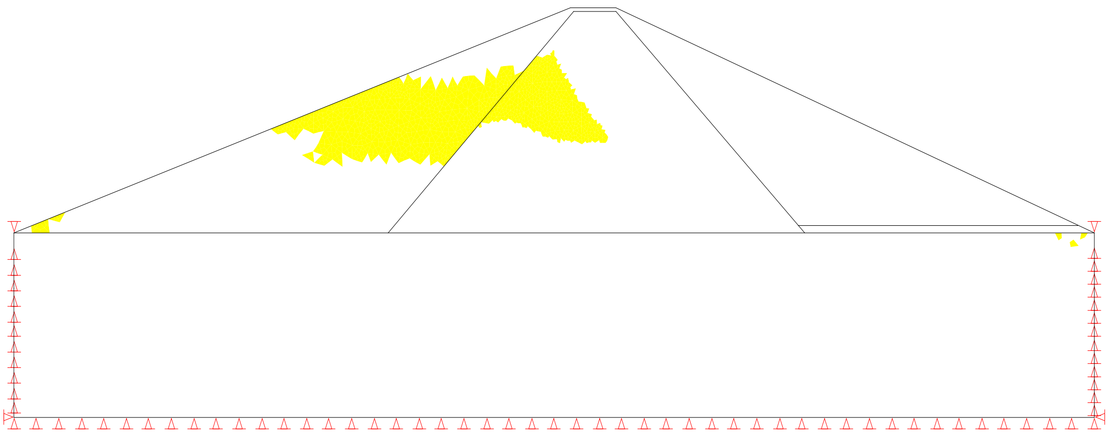
Figure 11: Earthquake response along two sections through the dam at maximum water level, at time 60 sec, with a horizontal drain 2 m thick.



(a) Minimum water level



(b) Normal water level



(c) Maximum water level

Figure 12: Propagation of liquefaction zones through the dam at time 600 sec, under a maximum horizontal acceleration of 0.2 g, with a horizontal drain 2 m thick.

Table 4: Computed values of maximum pore water pressure through the dam with a horizontal drain 2 m thick under a maximum horizontal acceleration of 0.2g.

Time (sec)	Maximum pore water pressure (kPa)		
	Minimum W.T.L (m)	Normal W.T.L (m)	Maximum W.T.L (m)
10	742.5	880.3	1027.3
60	746.3	882.8	1029.7
600	833.3	960.2	1115.1

Conclusions

1. The increase in the input acceleration delays the liquefaction occurrence when the value of acceleration is below 0.2g.
2. The time required for liquefaction to take place at the upstream shell is less than that required for other zones due to low overburden pressure at this part. An observation made on the computed effective stress time history indicates that the dam foundation dose not liquefy (where liquefaction means zero effective stress).
3. There is attenuation of the acceleration to some degree depending on the amplitude of the input horizontal acceleration. The attenuation ranged between (20-44) % for all cases. This means that the soils of the dam body represent soft materials. In contrast, in rockfill dams, during earthquake acceleration is magnified as it propagates from the base to the top of the dam.
4. The value of pore water pressure generated at the base of the core is greater than that in the upper parts of dam. The pore water pressure increases with the increased of earthquake acceleration.
5. The horizontal and vertical effective stress continue to decrease during the period of analysis 600 sec. which indicates that the soil continue to weaken during this period. The horizontal displacement increases with depth of the point from the crest and the largest horizontal displacement will be at the base of the dam at time 60 sec.
Also, the maximum horizontal displacement decreases by about 37%, 45% and 49% when using a horizontal drain 2 m thick at the downstream under a peak acceleration of 0.05g, 0.1g and 0.2g, respectively.
6. The pore water pressure decreases with the presence of horizontal drain by about 2.4 -3.8 %.

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استجابة السدود الترابية لإثارة الهزات الأرضية – سد خاصة جاي كدراسة حالة

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الخلاصة

في هذا البحث أجري تحليل ديناميكي لسد ترابي متعدد المناطق معرض لإثارة هزة أرضية حيث تم حساب ضغط ماء المسام و الاجهادات المؤثرة و الإزاحات. أستعملت طريقة العناصر المحددة و أعتمد برنامج الحاسبة Geo-Studio في التحليل من خلال برنامجيه الفرعيين SEEP/W و QUAKE/W. و قد أعتمدت دراسة الحالة لسد خاصة جاي الذي يقع على نهر خاصة جاي شمال العراق و يتألف من سدة ترابية متعددة المناطق بطول 3.34 كم. الهزة الأرضية المنتخبة للتحليل هي هزة السنترو ذات الفترة الزمنية 10 ثواني و بقيم تعجيل أقصى مختلفة، و أختير زمن تحليل مقداره 600 ثانية و بمراحل زمنية (Δt) تساوي 0.05 ثانية لدراسة تصرف التربة بعد فترة زمنية من توقف الهزة الأرضية حيث تم شمول فترة الاهتزاز الحر بالدراسة. و قد وجد أن قيمة ضغط ماء المسام المتولد عند قاعدة لب السد أعلى من قيمتها في الأجزاء العليا من السد. ان الاجهادات المؤثرة الأفقية و الشاقولية تستمر بالتناقص خلال فترة التحميل البالغة 600 ثانية مما يؤشر لأن التربة تستمر بالوهن خلال تلك الفترة، بينما تزداد الازاحة الأفقية مع عمق النقطة من قمة السد حيث تكون أكبر إزاحة أفقية عند قاعدة السد في زمن 60 ثانية و يحدث تلاشي للتعجيل إلى درجة معينة اعتمادا على التعجيل الأفقي المدخل. إن الإزاحة الأفقية القصوى تقل بنسب 37% و 45% و 49% عند استعمال مبزل أفقي بسمك 2 م في مؤخر السد تحت تأثير تعجيل أقصى للهزة مقداره 0.05 ج و 1 ج و 0.02 ج على الترتيب.