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SEISMIC RESPONSE OF AN ASPHALTIC CONCRETE CORE EMBANKMENT DAM

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ABSTRACT

In this paper a typical 115m high asphaltic concrete core rock fill dam is analyzed against maximum credible earthquake motion using two approaches; Newmark based models with equivalent linear method and nonlinear method. Nonlinear model is based on Mohr-Coulomb model with mobilized strength parameters. Results of analyses show greater values of accelerations in the crest of the dam in the equivalent linear method respect to nonlinear method. Deformation pattern of dam body in the nonlinear dynamic analysis is different from equivalent linear method in such a way that asphaltic core has less settlement than rock fill material especially at near crest. Plastic deformation are seen in both asphaltic core and surrounding area (fine transition zone), but developed shear strain in the filter material is much higher than asphaltic core. Shear strain and volumetric strain in a region near the crest elevation increase during the dynamic analysis, which causes a deflection of asphaltic core toward the upstream face. Both two kinds of analyses show development of crack in the upper part of the asphaltic core.

Keywords: Equivalent linear method, nonlinear model, dynamic Analysis, asphaltic concrete core, embankment dam

INTRODUCTION

In some area, which sufficient clay material is not available or rainy weather causes to an inappropriate condition for compaction of clay core, one of the best alternatives for preventing seepage through embankment dam is to use asphalt mixture in the dam body as an impervious material in the central or upstream part of the dam. The asphaltic concrete is virtually impervious, flexible, resistant to erosion and aging, workable and compactable and offers joint less core construction.

The seismic analysis and fundamental design based on true displacement mechanism has been rarely dealt with so far. In fact behavior of asphaltic concrete core embankment dams during the earthquake motions are not known properly; however in area with high seismicity condition, it is required to evaluate the response of this kind of dams against earthquake motion. For instance, real reaction of each section of the dam body and their interaction should be identified.

Valstad et al (1991) analyzed the Storvatn dam using a Newmark based method to derive the earthquake induced permanent displacements in the critical sliding surfaces and conclude that in an area of moderate seismicity the Storvatn dam has an adequate margin of safety. For a high seismic hazard area however, they concluded that the Storvatn dam design could function adequately if the outer slopes were decreased.

Dynamic analysis and design of Ceres dam, an asphaltic concrete core dam in South Africa, was reported by Meintjes and Jones (1999). They used pseudo-static analysis to fix the geometry of the

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dam. This was followed by a Newmark based method to derive the earthquake induced permanent displacement. They concluded that the design is acceptable.

Gurdil (1999) analyzed the dynamic behaviour of the asphaltic concrete core Kopru dam for both DBL and MCL hazard levels. In the determination of stress dependent modulus of elasticity and poisson's ratio, He used the hyperbolic model in the static analyses. In the dynamic analysis an equivalent linear method was used. He also took the hydrodynamic effects using the added mass method. He considered the stresses and the strength of the core and concluded that cracking may occur in the core but the self-healing mechanism of asphalt will stop the post earthquake seepage within a short time.

Ghanooni and Mahin Roosta (2002) reported their seismic analysis and design feathers for an asphaltic concrete core rock fill dam. They used equivalent linear method and nonlinear method for dynamic analysis. They concluded that in the nonlinear analysis, transition material in either side of the core became plastic and experienced large deformations, but asphaltic core stayed elastic. Also the Newmark based method using equivalent linear dynamic analysis showed extensive cracking in the asphaltic core.

In this paper a typical 115m high asphaltic concrete core rock fill dam is considered. It is another alternative for Narmashir rock fill dam in Iran. The different stages of construction and impounding are analyzed using an elasto-plastic model. This analyze provide initial stresses for seismic response analysis. The dynamic analysis is then followed using a) Equivalent linear method followed by Newmark based analysis to derive the earthquake induced permanent displacements and b) Nonlinear elasto-plastic method, in which irreversible displacement calculates during the analysis.

Dynamic analyses are carried out using the finite difference code FLAC (Itasca, 1998). The program is adjusted to incorporate the equivalent linear method in the dynamic case; also some routines (fish functions) are provided in the nonlinear analyses which will be explained in the material model part.

GEOMETRY AND MESH OF THE EMBANKMENT

Figure 1 shows the cross section of the 115 m height embankment dam. The vertical asphaltic concrete core has a thickness of 1.0 m, the fine transition either side of the core have a width of 1.5 m and the width of the coarse transition is 3.0 m. The slope of the upstream and the downstream shells are 1.V:1.6H. The dam body has lain on strong rock layers.

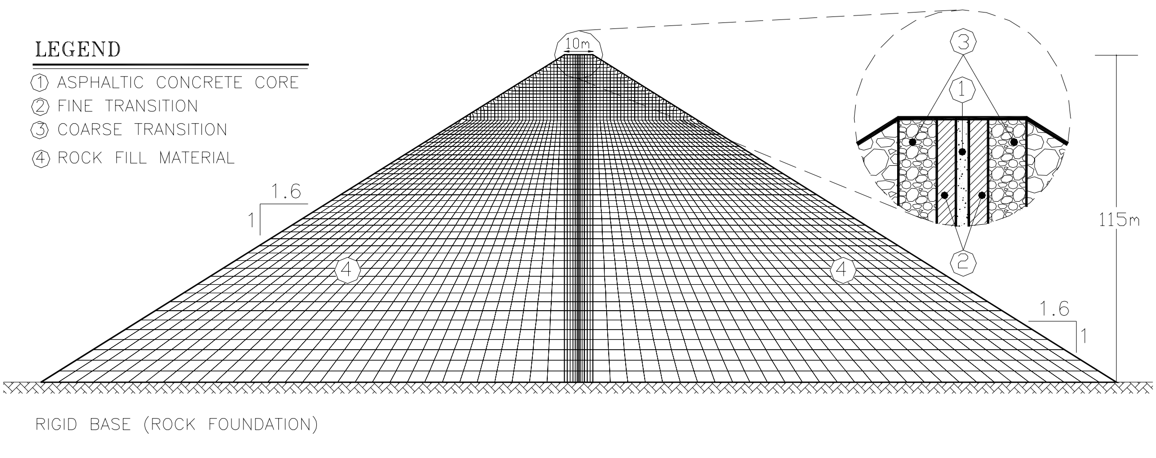

Figure 1. Finite difference model and different parts of dam

The dam section is discretized into a plane strain finite different grid. Based on Kuhlemeyer and Lysmer's study (1973), for accurate representation of wave transmission through the model, maximum dimension in the grids is considered to be less than one-tenth of the earthquake wavelength. Beside of this criterion, due to stress and strain concentration in the upper part of the dam, finer elements are used near the dam crest. Also asphaltic core is modeled using two elements in the thickness.

INPUT MOTION CHARACTERISTICS

Dynamic analyses are carried out for an extreme earthquake with a peak ground acceleration of 0.54g. Figure 2 shows the acceleration time history, pseudo spectral acceleration and the Fourier amplitude spectrum of the input motion. It is shown that maximum response of a system with a single degree of freedom is about 1.3g and majority of earthquake has frequency less than 10 HZ.

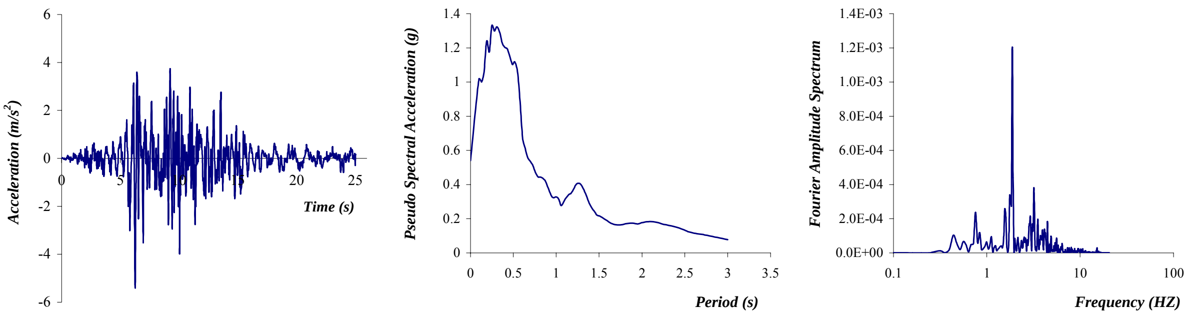

Figure 2. a)Acceleration time history, b) Pseudo spectral acceleration (5% damping) and c) Fourier amplitude spectrum

MATERIAL MODELING AND METHODS OF ANALYSIS

The dynamic analyses use two different material models; first equivalent linear model and second nonlinear model. These two models and their parameters are explained in the following paragraphs.

Equivalent linear method

In this model the dynamic behaviour of the material is described through the following parameters:
- Initial shear modulus or shear modulus in small shear strain, Gmax
- The Poisson's ratio
- The decrease of secant modulus G with increasing shear strain γ
- The hysteretic damping ratio, ξ, which is an increasing function of the amplitude of shear strain γ.

In the rock fill and transition material, shear modulus is estimated as a function of effective confining pressure $\sigma'_o = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$, (Ishihara 1986). Maximum shear modulus, Poisson's ratio and porosity (n) for different part of the dam body is shown in Table 1. In the shear modulus formula, e is void ratio and is related to porosity, n:

$$e = \frac{n}{1-n} \tag{1}$$

Decrease of secant modulus (G) and increase of damping ratio (ξ) with increasing shear strain (γ) are shown in Figure 3. Changes of these parameters for rock fill and transitions are based on the studies of Kokusho and Esashi (1981) and Seed and Idriss (1970).

Amount of damping ratio in the asphaltic concrete core is much higher than other part of the dam body. In this thin part of the embankment, damping ratio varies from .18 to .53 (Nakamura et al, 2004). Beside of this phenomenon, reduction of shear modulus in the asphaltic concrete core is less than other materials.

Table 1. Material properties in the dynamic analyses

Propertie s	Asphaltic concrete core	Transitions	Rock fill
γ (kN/m^3)	24.20	21.50	22.00
n	0.03	0.20	0.30
ν	0.45	0.3	0.25
Gmax	2.5 GPa (Nakamura et al, 2004)	$8400\frac{(2.17-e)^2}{1+e}(\sigma_0')^{0.6}$ (Kokusho and Esashi, 1981)	$13000\frac{(2.17-e)^2}{1+e}(\sigma_0')^{0.55}$ (Kokusho and Esashi, 1981)
φ_p (^{o})	28	38	46
ψ_p (^{o})	0 - 4	5	8
C (MPa)	.36	-	-
T (MPa)	.36	-	-

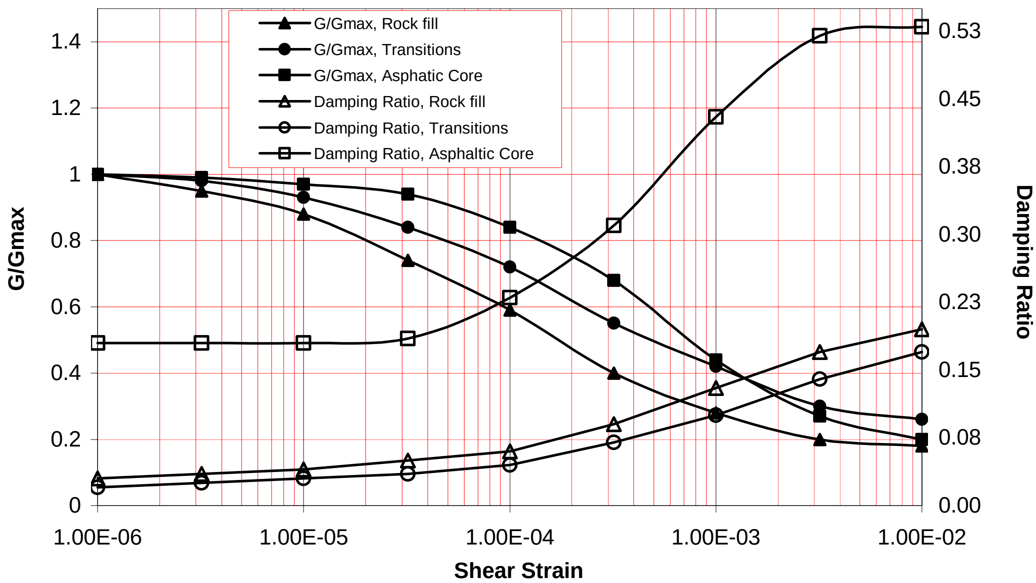

Figure 3. Modulus reduction curves and variation of damping ratio of materials

Non-linear elasto-plastic model

The nonlinear material model used in this study is a strain hardening/softening model (SS model) based on FLAC Mohr-Coulomb model (Itasca, 1998). It has the possibility that the cohesion, friction, dilation and tensile strength may harden or soften after the onset of plastic yield. In this research hardening flow rule is applied in the rock fill and transitions zones; The frictional hardening relation is the one proposed by Vermeer & de Borst (1984):

$$\sin\varphi_m = \frac{2\sqrt{\gamma\gamma_p}}{\gamma+\gamma_p}\sin\varphi_p\,,\quad \gamma \le \gamma_p$$
$$\varphi_m = \varphi_p\,,\quad \gamma > \gamma_p \tag{2}$$

Where φ_p = Ultimate friction angle

φ_m = mobilized friction angle
γ = plastic shear strain, and
γ_p = plastic shear strain at ultimate friction angle.

Figure 4 gives the relationship between mobilized friction angle and corresponding plastic shear strain for rock fill materials. Ultimate friction angles in rock fill and transition zones are 46° and 38°, respectively.

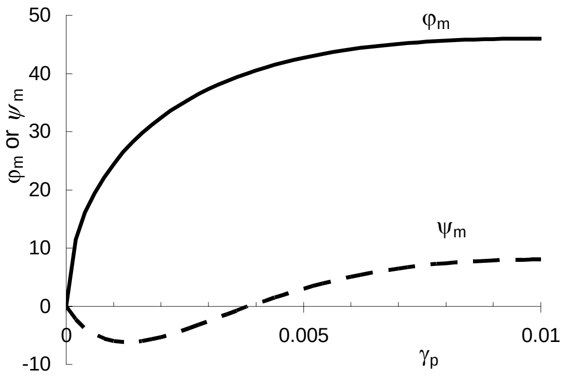

Figure 4. Mobilization of friction and dilation in nonlinear analysis

The value of Gmax is also derived from the expression given in previous section (Table 1). With concept of mobilized friction angle and initial shear modulus, backbone curve is produced in the nonlinear analysis.

Dilation angle is mobilized based on stress-dilatancy theory, which was defined by Row (1963):

$$\sin\psi_m = \frac{\sin\varphi_m - \sin\varphi_{cv}}{1 - \sin\varphi_{cv}\sin\varphi_m} \tag{3}$$

Where ψ_m is mobilized dilation angle and φ_{cv} is friction angle at constant volume (or when mobilized dilation is zero). φ_{cv} is defined as follows:

$$\sin\varphi_{cv} = \frac{\sin\varphi_p - \sin\psi_p}{1 - \sin\varphi_p\sin\psi_p} \tag{4}$$

And ψ_p is peak dilation angle. Peak dilation angle for rock fill and transition zones are 8° and 4°, respectively. By considering above definitions, dilation angle is related to plastic shear strain.

Based on the above formula, dilation angle in very small plastic shear strain is too large and negative; to remove this drawback, the definition is modified by following equations (Søreide et al, 2002):

$$\sin\hat{\psi}_m = \sin\psi_m\left(\frac{\sin\varphi_m}{\sin\varphi_p}\right)^n \tag{5}$$

Power n controls the shape of the mobilized dilation angle and in this study is set to 5. Figure 4 shows changes of mobilized dilation angle with plastic shear strain in rock fill material. It can be seen from this figure that mobilized dilation angle starts from zero at zero plastic shear strain, then it has negative value (contraction behavior) and changes to positive one (expansion behavior) with increase in the plastic shear strain.

Mohr-Coulomb material model is considered in the asphaltic concrete core. Model parameters comprise friction angle, dilation angle, cohesion and tensile strength, which are shown in table 1.
In the asphaltic core, dilation causes an increase in permeability due to opening of small fissures, although no visible cracks may appear. Due to volume expansion about 1-2 %, the permeability coefficient in the asphaltic mixture shows an increase by a factor of $10^3 - 10^5$ (Hoeg, 1993). Since dilation is volumetric change due to shear deformation, dilation angle has great effect on the permeability of asphaltic core; for instance dilation angle in the asphaltic core of the embankment is considered 0, 1, 2, 3 and 4^o for five part of the height of dam (While confining pressure increases, dilation angle decreases).

The amount of viscous damping is taken as minimum values of Figure 3; e.g. 0.03, 0.04 and .18 for rock fill, transitions and asphaltic core, respectively. These values are added to the hysteretic damping which develops during cycles of loading and unloading due to nonlinearity in the material model.

RESULTS OF DYNAMIC ANALYSES

Using above mentioned material models, the dynamic response of the dam to maximum earthquake level (MCL) in the steady seepage condition (full reservoir) were analyzed. Because of high value of permeability in the rock fill and transition, no generation of excess pore water pressure was assumed during earthquake excitation. In the following sections, results of equivalent linear and nonlinear method are compared with each other. It should be mentioned that the frequency content of the selected acceleration time history influences the response of the dam as well as the resulted developed displacements.

Crest acceleration response

Figure 5 represents horizontal crest acceleration in both equivalent linear and nonlinear analyses. It can be seen that horizontal acceleration amplitudes in the linear analysis are greater than nonlinear analysis; The maximum acceleration experienced in the equivalent linear method and nonlinear method is 1.5 g and 1 g, respectively. However, number of peaks in the accelerations response of the crest in the nonlinear analysis is more than the linear analysis. In the nonlinear analysis, due to yielding surface, acceleration in each element becomes less than a critical acceleration; In fact, differences of these accelerations produce permanent displacement in each element.

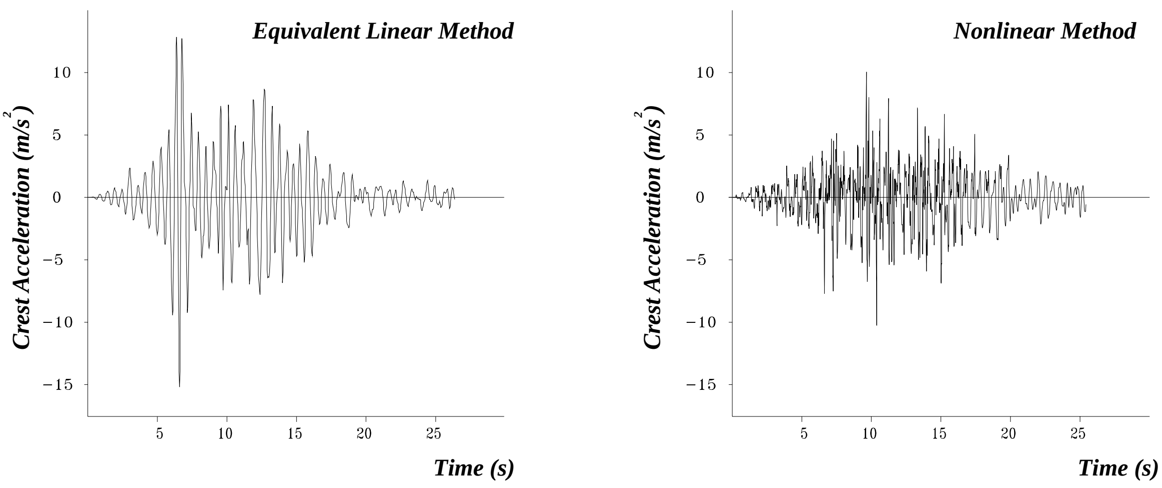

Figure 5. Crest accelerations histories in equivalent linear and nonlinear dynamic analyses

Displacement field

In equivalent linear method, displacement is determined from the Newmark approach (1965). For instance, average seismic coefficient for critical slip surfaces at different heights of the embankment is derived from equivalent linear method. The permanent displacements are then determined by double integration between average seismic acceleration and yield acceleration, as was suggested by Newmark.

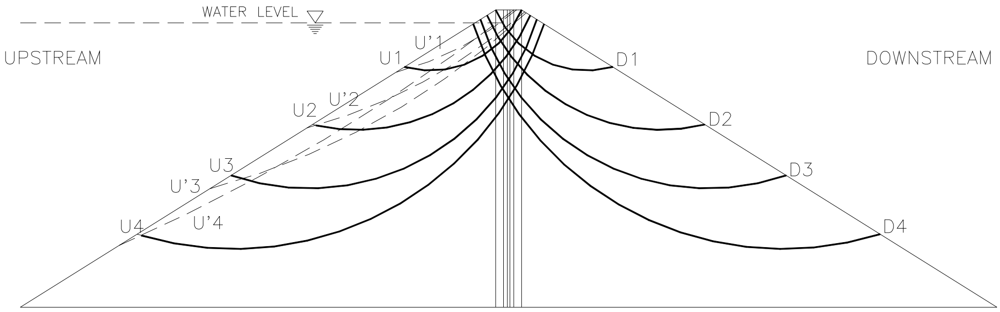

Figure 6. Sliding masses in the Newmark analysis.

Figure 6 shows the selected upstream and downstream slip surfaces. For the upstream side two sets of slip surfaces are chosen. The first set is chosen in such a way to cross a substantial amount of the core and the second set is chosen as the most critical surface (i.e. with the lowest factor of safety) at each level. The yield acceleration coefficient (Kc) for each slip surface is derived using the pseudo static analysis. Kc is horizontal seismic coefficient, in which safety factor of stability of each sliding mass becomes 1 (each slide starts to move). History of average acceleration response of each slip surface is calculated from acceleration response of the nodes inside the sliding masses. Figure 7 gives the average acceleration time histories and yield accelerations of two slides in both side of the dam.

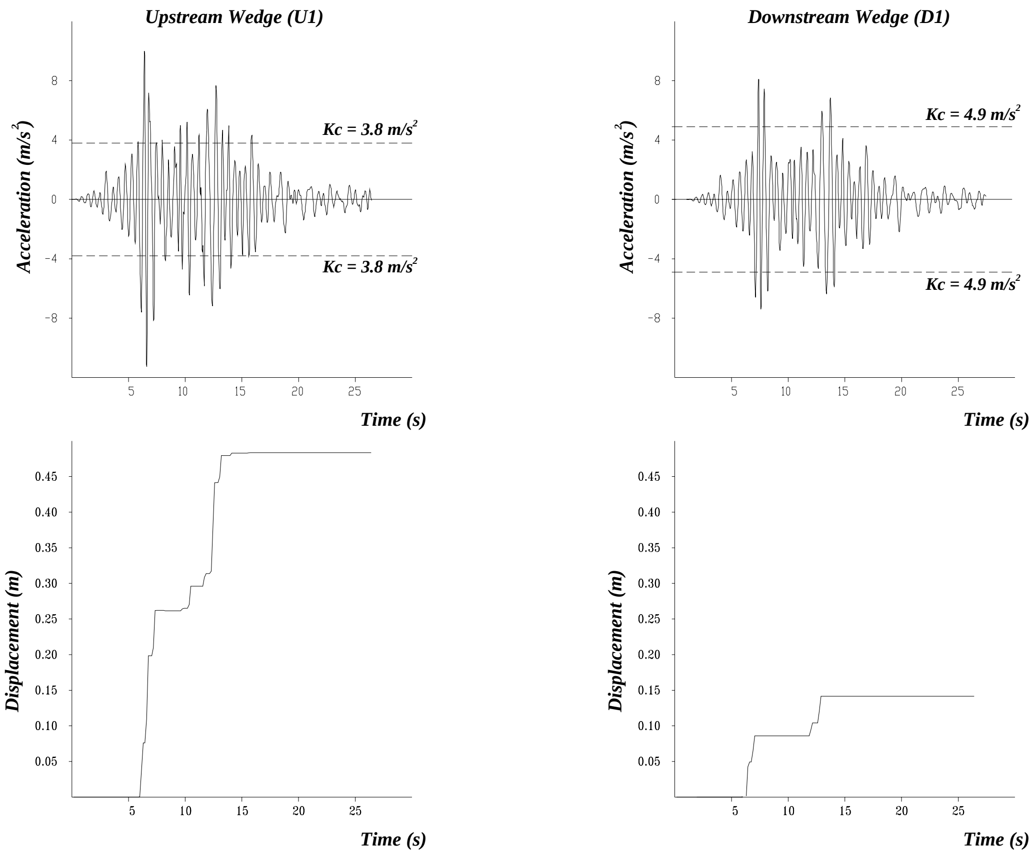

Figure 7. Acceleration histories from equivalent linear method and permanent displacement from Newmark concept

Displacement time histories of slides U1 and D1 from Newmark method is also shown in Figure 7. Maximum jump in the displacement develops during 5 to 15 seconds of the input motion. These histories of displacement are maximum displacement of either side of acceleration response (Figure 7).

Table 2 gives the results of the analysis for all sliding masses; in this table, K_m is maximum seismic coefficient of each sliding mass during earthquake motion. It can be seen from table 2 that maximum movement develops in the upper sliding masses. In fact, blocks U1, U'1 and U'2 are more critical than other slides. These displacements are too large and comparable with asphaltic core thickness. Thus Newmark method shows large displacements in slip surfaces and widespread cracking of the core.

Table 2. Results of Newmark based permanent displacement method

Slip surface	U1	U2	U3	U4	D1	D2	D3	D4	U'1	U'2	U'3	U'4
K_c (g)	0.38	0.32	0.32	0.28	0.49	0.41	0.4	0.35	0.36	0.26	0.24	0.23
K_m(g)	1.12	0.82	0.53	0.38	0.81	0.68	0.51	0.36	1.17	0.91	0.66	0.45
Disp. (cm)	48	32	5.2	1	14	3.7	0.23	0	64	79	36	18

In the nonlinear analysis, permanent displacement is determined through the analysis directly. In fact, postulation of stresses from yield criterion during dynamic analysis causes plastic shear strains and permanent displacement. Figure 8 shows the deformation field of the embankment caused by earthquake. The crest settlement of the embankment is about 1.25 m but the settlement of the core is very small. It can be seen form contours of y-displacement that more settlements occurs in the fine transition zone. In fact due to differences of strength and deformability of core and transitions, shear strain concentrates between them. Grid deformation in figure 8 can show this situation very clearly. Horizontal displacements are displayed in this figure, too. Maximum horizontal displacements are 1.25 and 0.5 m in the upstream and downstream direction, respectively. These contours show that depth of sliding is little, which is usual in the cohesion less materials. Upstream horizontal displacement contours are comparable with prescribed slides in the Newmark concepts explained in previous section (Figure 6, slide U'1 and U'2).

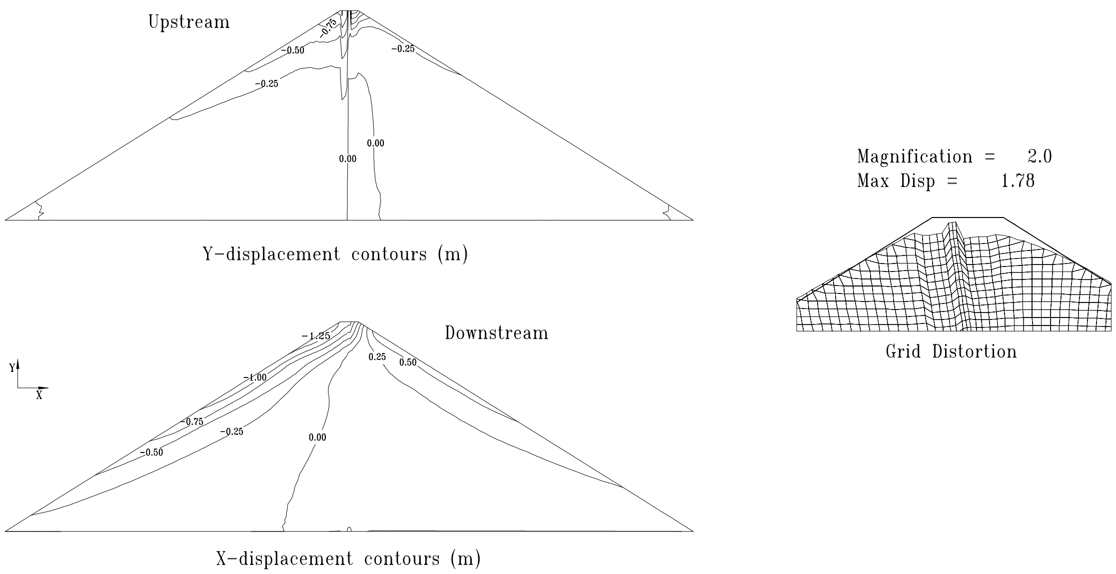

Figure 8. Displacement field at end of nonlinear dynamic analysis

Figure 9 gives shape of deformation of asphaltic concrete core during some times of the dynamic analysis. It is obvious that majority of deformation occurs in the time period of 5 to 15 seconds and maximum deflection exists in 15 to 20 m bellow the top of the dam. In this area, plastic shear strain develops during analysis and causes increase in the volume of the element due to dilation. Time histories of shear strain and volumetric strain of element M in Figure 9 is shown in Figure 10. Shear and volumetric strain in the initial 5 seconds of the analysis are very small and yielding occurs after this period of time. Due to formation of sliding masses, volumetric and shear strains increase until end of dynamic analysis.

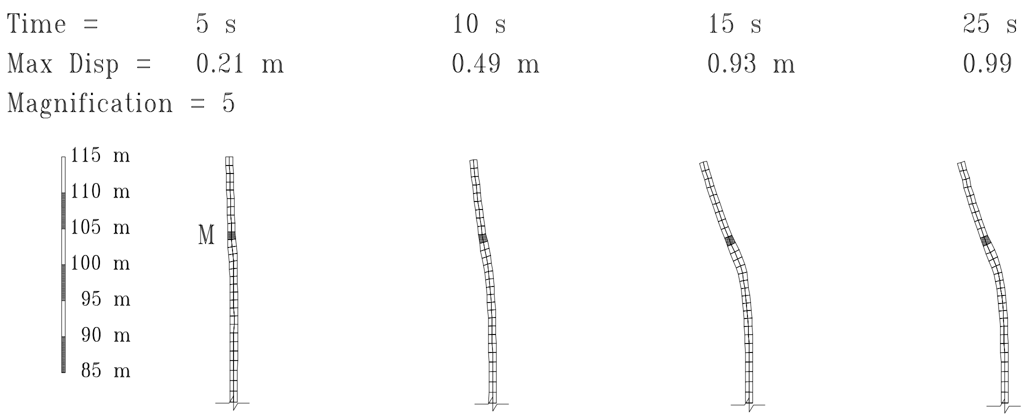

Figure 9. Shape of deformation of asphaltic concrete core

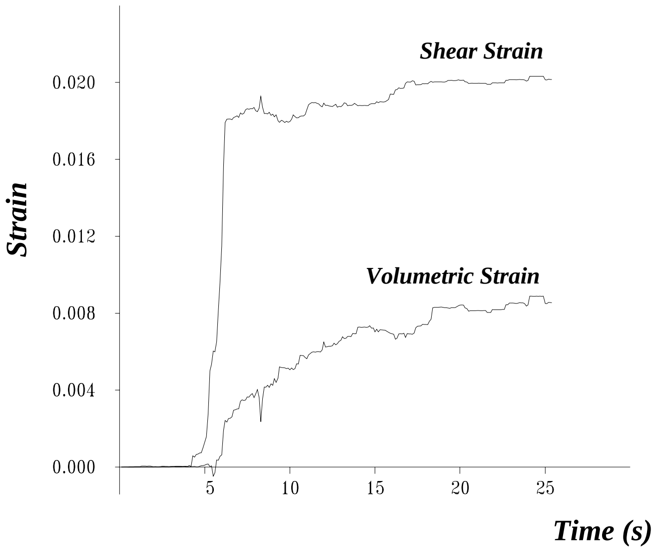

Figure 10. Histories of shear and volumetric strain of a point in the asphalt core

Maximum volumetric strain and shear strain in the height of the asphaltic concrete core are represented in Figure 11. Since the zone of maximum curvature experiences tensile and shear yielding during dynamic analysis, Shear strain reaches to 4 % in this region. In addition to increase of shear strain, maximum volumetric strain reaches to about 1 % in this area; thus porosity of asphaltic core changes from its initial value (3 % in Table 1) to more than 4 %; so permeability of this zone could increase to about 1000 times (for example from 10-10 to 1e-7 cm/s) (Hoeg, 1993).

Like equivalent linear and Newmark method, the nonlinear dynamic analysis shows that crack could happen in the asphaltic core. But crack elevation is near the crest of dam and so in the worst situation (maximum flood level), water pressure can cause a little increase in the flow through the dam body. Also for preventing crack development in the asphaltic core, one can design a more ductile core using especial bitumen or increase the bitumen content in the mix design.

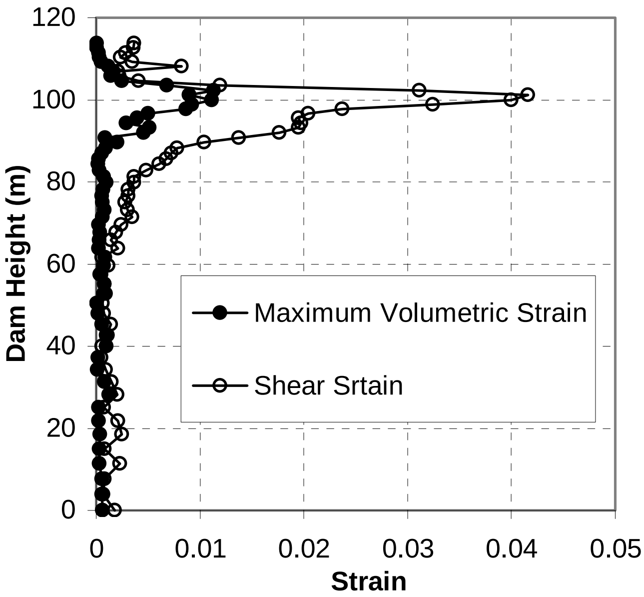

Figure 11. Variation of shear strain and maximum volumetric strain with height of asphaltic core.

CONCLUSION

This study was focused on a comparison between the behavior of asphaltic concrete core in equivalent linear and nonlinear method. The following conclusions are as follow:

- Acceleration magnification in nonlinear analysis is smaller than linear analysis.
- In proposed nonlinear model, maximum displacement of embankment dam is larger than that of equivalent linear method and using Newmark concept.
- Large differential settlement occurs between the core and the surrounding transition material. In fact shear strain concentrates in the fine transition zones.
- The Newmark based method using equivalent linear dynamic analysis shows extensive cracking of the core.
- Due to development of sliding mass in the nonlinear model, top of the asphaltic core deflects to the upstream side of the dam. Actually tensile and shear yielding occur in the upper part of the core which cause some region to be dilated and produced minor cracks.

It has to be mentioned that in the present study the effects of 3-D canyon shape, hydro-dynamic effects and radiating boundary condition have not been investigated.

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