Evaluation of Eulerian and Lagrangian Method in Analysis of Concrete Gravity Dam Including Dam Water Foundation Interaction

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***Abstract*—**Because of the reservoir effect, dynamic analysis of concrete dams is more involved than other common structures. This problem is mostly sourced by the differences between reservoir water, dam body and foundation material behaviors. To account for the reservoir effect in dynamic analysis of concrete gravity dams, two methods are generally employed. Eulerian method in reservoir modeling gives rise to a set of coupled equations, whereas in Lagrangian method, the same equations for dam and foundation structure are used.

The Purpose of this paper is to evaluate and study possible advantages and disadvantages of both methods. Specifically, application of the above methods in the analysis of dam-foundation-reservoir systems is leveraged to calculate the hydrodynamic pressure on dam faces. Within the frame work of dam- foundation-reservoir systems, dam displacement under earthquake for various dimensions and characteristics are also studied. The results of both Lagrangian and Eulerian methods in effects of loading frequency, boundary condition and foundation elasticity modulus are quantitatively evaluated and compared. Our analyses show that each method has individual advantages and disadvantages. As such, in any particular case, one of the two methods may prove more suitable as presented in the results section of this study.

***Keywords*—**Lagrangian method- Eulerian method- Earthquake- Concrete gravity dam.

I. Introduction

HYDRODYNAMIC pressure on the upstream face of the concrete dams under the effect of earthquake is one of the most important parameters, in planning a structure in earthquake zone. As a result, the research must be able to evaluate the response of dam with consideration of dam's interaction with reservoir and its foundation. This problem has been studied vastly with different researchers.

The first research on the analysis of concrete gravity dam has been done by Westergaard [1] in 1930 and its analysis response for hydrodynamic pressure on the dam face was clear. But Kotsubo [2] showed that Westergaard's finding is valid just when the harmonic excitation period is smaller than the fundamental natural reservoir period. Hilborn [3] also studied the effect of the length of reservoir on hydrodynamic pressure. The findings of Jacobson [4] support the above researches. Werner [5] showed that the responses are not sensitive to the length of the reservoir. And the Bustamante [6] studied the result of the reservoir's length for a range of periods of excitation greater than the fundamental natural period of reservoir. He also studied the effect of surface waves under harmonic excitation and its ignorant error. Zangar [7] determined hydrodynamic pressure for various shapes of the upstream face of dam. In a paper in 1961, Kotsubo [8] found and presented hydrodynamic response of a reservoir and arch dam under earth harmonic movement. In the way Chopra [9] presented the response of dam under the horizontal and vertical acceleration of the earth. Chopra [10] also studied on dam-reservoir interaction and its semi infinite foundation.

There were a lot of other researches which studied the linear behavior of the dam-reservoir system, including nonlinear behavior of the dam under pressure and also cavitation. In each research, different modeling methods are presented which are divided into two main groups. In first method which is called Eulerian, pressure is the main unknown parameter in reservoir nodes. In the second method the main unknown parameter displacement of nodes, this is called lagrangian method. Each of the methods contain some advantageous and disadvantageous. In this paper we present hydrodynamic pressure and dam crest point displacement with consideration of its interaction with reservoir and foundation.

II. Formulation of Lagrangian - Lagrangian Method

Concrete gravity dam- reservoir- foundation systems are three dimensional but are idealized as two dimensional sections in planes normal to the dam axis. Application of the lagrangian (the standard displacement-based finite element) method over the domains of dam, foundation and water produces the following global matrix equation [11]:

$$M\ddot{u} + C\dot{u} + Ku = F(t) \tag{1}$$

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Where M = mass matrix, C = damping matrix, K = stiffness matrix, F = dynamic load vector and U is vector of unknown nodal displacements and a super dot indicates a material time derivative.

A. Concrete gravity dam and foundation

Concrete of dam and foundation is assumed linear, elastic, isotropic and homogeneous. The standard finite element discretization leads to following element matrixes and vectors [11]:

$$M_c = \int \rho_c N^T N \; dV \tag{2}$$

$$K_c = \int B^T DB \; dV \tag{3}$$

$$D = \frac{E}{1-\nu^2}\begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \dfrac{1-\nu}{2} \end{bmatrix} \tag{4}$$

$$F(t) = -M\begin{Bmatrix} 1 & 0 \\ 0 & 1 \\ 1 & 0 \\ . & . \\ . & . \end{Bmatrix}\begin{Bmatrix} \ddot{U}_g \\ \ddot{V}_g \end{Bmatrix} \tag{5}$$

Where N and B are shape functions and strain transformation matrixes, respectively. ρ_c is the dam concrete mass density, $\ddot{U}_g$ and $\ddot{V}_g$ are the free field ground acceleration, D is the tangent constitutive matrix. E_c is young's modulus of dam plain concrete and ν is poisson's ratio.

Structural damping matrix is defined to be stiffness – mass proportional as

$$C_c = \alpha K_c + \beta M_c \tag{6}$$

where α and β is determined by specifying two described damping ratio(ξ) at two given frequency(ω).

$$\begin{Bmatrix} 2\xi_1\omega_1 \\ 2\xi_2\omega_2 \end{Bmatrix} = \begin{bmatrix} \omega_1^2 & 1 \\ \omega_2^2 & 1 \end{bmatrix}\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} \tag{7}$$

Inclusion of foundation interaction introduces some flexibility at the base of dam and provides an additional means for energy dissipation through radiation. For the purpose of computation the foundation energy dissipation following damping matrixes for shear and longitudinal waves [11]:

for longitudinal waves on S5:

$$C_{R5} = \sqrt{\frac{E_f(1-\nu_f)}{(1+\nu_f)(1-2\nu_f)\rho_f}}\int_{s_5} N^T n^t n \; N.ds \tag{8}$$

for shear waves on S_5:

$$C_{R5} = \sqrt{\frac{E_f}{2(1+\nu_f)\rho_f}}\int_{s_5} N^T n^t n \;\; N.ds \tag{9}$$

Where ρ_f, E_f and ν_f are foundation mass density, young's modulus poisson's ratio, respectively. And n is the normal direction of foundation boundaries.

B. Reservoir

The standard displacement-based fluid model is used in this method. The fluid is assumed linear, inviscid and irrotational. In this section the water element matrixes and vectors are defined and the boundary conditions to water are incorporated in to the finite element equations.

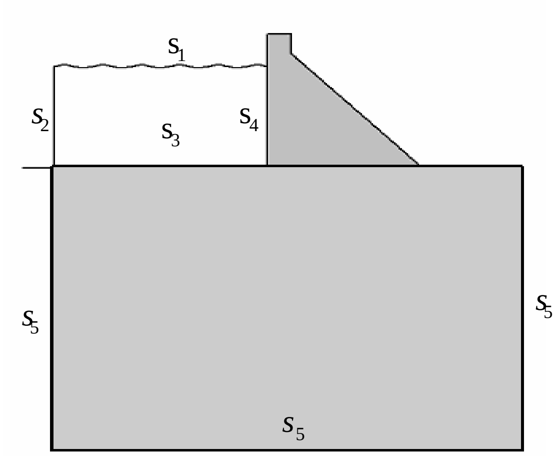

Fig. 1: dam- reservoir- foundation system and its boundaries

The water element mass matrix and dynamic load vector are identical to their counterparts for dam given in equations (1) to (4) except for the use of the water mass density (ρ_w). Since the fluid is assumed inviscid, its strain energy is due only to deformational modes with volumetric strain. But the ordinary element stiffness matrix has mostly zero-energy deformational modes, and hence, the assembled water stiffness matrix is expected to be singular. This is remedied by enforcing the irrotationally condition and including the linearized small amplitude wave boundary condition at the free surface (on S1) [12].

$$S_f = \rho_w.g\int_{s_3} N_w{}^T.N_w.\mathrm{ds} \tag{10}$$

Irrotationality is included a penalty formulation that leads to an additional term is the element stiffness matrix given by [12]:

$$K_w = \int_V B^T D_w B \,\mathrm{dv} \tag{11}$$

$$D_w = \begin{bmatrix} K_B & 0 \\ 0 & \alpha \times K_B \end{bmatrix} \tag{12}$$

Where B is the nodal displacement-volume strain transformation matrix. KB is the bulk modulus of water, α is a penalty number and Nw is the fluid nodes` shape function.

Reduce integration is again used in integrating equation (11), as required by penalty methods [12].

Along the dam-water (on S_3) or foundation-water interface (on S_4) each water node occupies the same location as a dam or foundation node. For this purpose in these boundary nodes with 3 free degrees (2 vertical degrees and 1 horizontal free degree) are used [13].

Energy dissipation with water is due to radiation in the infinite upstream direction (on S_2) and absorption along the foundation interface (on S_3). Assuming one dimensional wave propagation in the foundation-water interface boundary in direction normal to S3 and in reservoir normal to upstream direction of S_2 (sommer feld condition) analytical explicit representation for pressures along these boundaries be obtained [11]

$$C_{R_1} = \rho_w . C_w \int_{S_1} N^T . N . ds \tag{13}$$

$$C_{R_2} = \frac{\rho_w . C_w}{\beta} \int_{s_2} N^T . N . ds \tag{14}$$

$$M = M_c + M_\omega , C = C_c + C_{f1} + C_{f2} + C_{\omega 1} + C_{\omega 2} ,$$

$$K = K_c + K_\omega + S_f \tag{15}$$

III. Formulation of Lagrangian - Eulerian Method

A. Concrete gravity dam and foundation

In this method application of lagrangian method over the domains of dam and foundation such as previous method produces the following global equation [14]:

$$M\ddot{u} + C\dot{u} + Ku = F(t) \tag{16}$$

Where M, C, K and F(t) are dam and foundation mass, damping, stiffness matrixes and dynamic force vector and calculate by same equations that said in previous section.

B. Reservoir

The hydrodynamic pressure based fluid model is used in this method. The equation of hydrodynamic wave propagation in reservoir is the following quasi-harmonic equation [14]:

$$\nabla^T K_B \nabla P - \rho_w \ddot{P} = 0 \qquad in \quad \Gamma \tag{17}$$

Where K_B is the bulk modulus of water, ρ_ω is the water mass density and P is the hydrodynamic pressure. Also Γ is the inside surface of reservoir.

For solving this matrix equation Galerkin method is used. Weak form of this equation is such as following equation [14]:

$$\int_\Gamma \nabla^T W K_B \nabla P \, \mathrm{d}\Gamma + \int_\Gamma W \rho \ddot{P} \, \mathrm{d}\Gamma - \int_{s_q} W K_B (\nabla P)^T n \, \mathrm{ds} = 0 \tag{18}$$

Where W is the weight function and S_q is the reservoir boundaries.

By introducing shape function (N_i), the hydrodynamic pressure in reservoir calculates by following equation:

$$p = \sum_{i=1}^{m} N_i P_i = N \, P \tag{19}$$

Where m is the number of nodes.

By selecting of shape function as weight function, equation (19) transforms in the form of following equation [14]:

$$\left[\int_\Gamma \nabla^T N \nabla N \, \mathrm{d}\Gamma\right] p + \left[\frac{1}{c^2} \int_\Gamma N^T N \, \mathrm{d}\Gamma\right] p - \int_{s_q} N^T \frac{\partial p}{\partial n} ds = 0 \tag{20}$$

$$H = \int_\Gamma \nabla^T N \nabla N \quad \mathrm{d}\Gamma \tag{21}$$

$$E_1 = \frac{1}{c^2} \int_\Gamma N^T N \quad \mathrm{d}\Gamma \tag{22}$$

Where H is quasi stiffness matrix and E_1 is quasi mass matrix of reservoir.

For calculating third sentence of equation (20), reservoir boundary conditions must be applied.

$$\int_{s_q} N^T \frac{\partial p}{\partial n} ds = \sum_{i=1}^{4} \int_{s_i} N^T \frac{\partial p}{\partial n} ds \tag{23}$$

Equation (23) calculates 1n 4 boundaries of reservoir by following equations:

1- For considering wave radiation in the infinite upstream direction (on S_2), in this method, following equation is used [14]:

$$\frac{\partial p}{\partial n} = \frac{-1}{c} \dot{p} \tag{24}$$

Substituting equation (24) in equation (23) produces following equation:

$$-\int_{s_1} N^T \frac{\partial p}{\partial n} ds = \underbrace{\left[\frac{1}{c} \int_{s_1} N^T N ds\right]}_{A_1} \dot{p} \tag{25}$$

Where A_1 is the radiation damping matrix of reservoir.

For considering abstraction effect along the foundation the jointed boundary of water and foundation (on S_3), the following equation is used [14]:

$$\frac{\partial p}{\partial n} = -\frac{1}{\beta c} \dot{p} \tag{26}$$

$$-\int_{s_2} N^T \frac{\partial p}{\partial n} ds = \underbrace{\left[\frac{1}{\beta c} \int_{s_2} N^T N ds\right]}_{A_2} \dot{p} \tag{27}$$

Where A_2 is the refraction damping matrix of reservoir.

For the purpose of computation of surface wave effect (on S_1), in this method following equation is used [14]:

$$\frac{\partial p}{\partial n} = \frac{-1}{g} \ddot{p} \tag{28}$$

$$-\int_{s_3} N^T \frac{\partial p}{\partial n} ds = \underbrace{\left[\frac{1}{g} \int_{s_3} N^T N ds\right]}_{E_2} \ddot{p} \tag{29}$$

Where E_2 is quasi mass matrix of reservoir because of surface wave.

For the purpose of considering dam-water and water-foundation interaction on these surfaces (S_3 & S_4), following equation must be used [14]:

$$\frac{\partial p}{\partial n} = -\rho \, \mathrm{a}_{\mathrm{ns}} \tag{30}$$

$$\mathrm{a}_{\mathrm{ns}} = \mathrm{n} \, \mathrm{a}_{\mathrm{s}} = \mathrm{n}\left(\overline{\mathrm{N}} \ddot{\mathrm{u}}_{\mathrm{tot}}\right) \tag{31}$$

$$a_{ns} = n\,\overline{N}\left(\ddot{u} + \ddot{u}_g\right) \tag{32}$$

Where ans is the structure acceleration, normal to interface boundary. n is the normal vector and as is the structure acceleration, $\overline{N}$ is the dam shape function, $\ddot{U}_{tot}$ is total acceleration of dam nodes, $\ddot{U}$ is relative acceleration and $\ddot{U}_g$ is ground acceleration.

$$-\int_{s_4, s_2} N^T \frac{\partial p}{\partial n} ds = \rho \underbrace{\left[\int_{s_4, s_2} N^T n \overline{N}\, \mathrm{ds}\right]}_{Q^T} \ddot{u}_{tot} \tag{33}$$

Where Q is interaction matrix.

Therefore, the equation of hydrodynamic wave propagation in reservoir is such as following equation [14]:

$$E\ddot{P} + A\dot{P} + HP = R^f \tag{34}$$

Where:

$$R^f = -\rho Q^T \ddot{u}_{tot}\,,\ \mathrm{A} = \mathrm{A}_1 + \mathrm{A}_2\,,\ E = E_1 + E_2 \tag{35}$$

IV. Analysis Method

Uniform condition is essential in compare two system modeling methods of Eulerian and Lagrangian. So visual C#.NET 2003 is used in this investigation that produces possibility of dynamic analysis of concrete dams under earthquake with system modeling by both methods. Nine node element for reservoir and eight node element for dam and foundation is used for both methods. Also newmark average acceleration method is used for solving dynamic's equibilirium equation.

V. Analysis of Pine Flat Dam

In this paper the response of the tallest, non-overflow monolith of Pine Flat dam in California, which is 122 m high, to horizontal and vertical earthquake is computed. The planner finite element model of Pine Flat dam monolith and its reservoir and foundation is shown in figure 2. A water depth of 116 m is considered the full reservoir condition, and the water has the following properties: unit mass, $\rho = 1000\ kg/m3$, bulk modulus, $K = 2.07*10^9$ kg/m2, and pressure wave velocity, $c_w = 1440 m/s$. The finite element model of reservoir consists of 12 isoparametric elements and it extends upstream a distance of 366 m, three times the dam height. The dam consists of 20 isoparametric elements. The concrete of dam has the unit mass of $\rho = 2500 kg/m3$, young's modulus of $E = 2.275*10^{10} kg/m3$, and poisson`s ratio of $\upsilon = 0.25$. The concrete of foundation has the unit mass of $\rho_f = 2500 kg/m3$, young's modulus of $E_f = 4.45*10^{10} kg/m3$, and poisson`s ratio of $\upsilon_f = 0.25$. Stiffness proportional damping in the dam provides a critical viscous damping ratio of 5% in the fundamental vibration mode of the dam. This study presents the response of Pine Flat dam monolith under the taft Lincoln school tunnel records from the 1952 Kern county, California. The S69E component is taken as the horizontal component. The peak acceleration of S69E and vertical components is 0.18g and 0.1g, respectively.

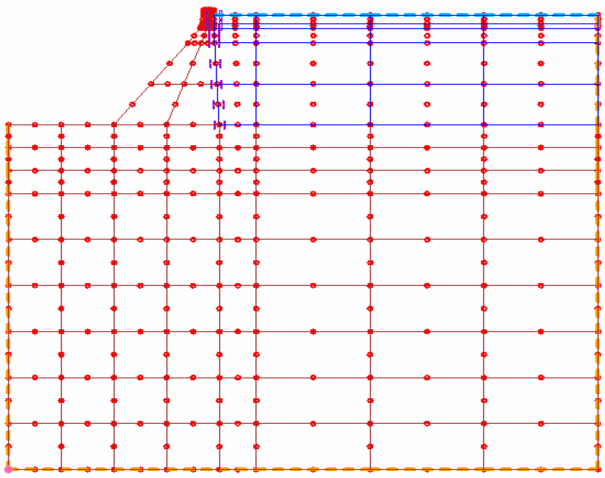

Fig. 2: finite element model of dam- reservoir- foundation system

VI. Numerical Results

One of the numerical examples, in analysis dam-reservoir system is the Westergaard`s response. For comparing, a reservoir with 300 m length anh 100 m height under harmonic excitation, according figure 3, is considered.

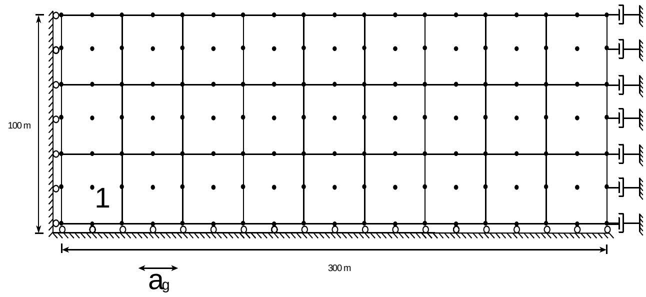

Fig. 3 :the finite element model of reservoir under harmonic excitation

Figure 4 show the hydrodynamic pressure in number 1 element center calculated by Westergaard`s equation and numerical Lagrangian and Eulerian methods. It is indicated that both methods have enough exaction.

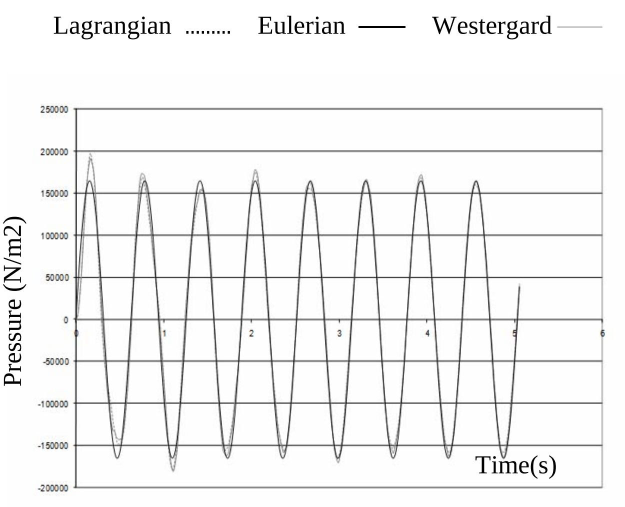

Fig. 4: hydrodynamic pressure in number 1 element center under harmonic excitation

In tables 1 to 6, the result of the different analysis of dam-water- foundation system is presented in which LH and LV show the horizontal displacement of the dam crest point with the Lagrangian method modeling under horizontal and vertical components of earthquake, and EH and EV are the response of the Eulerian method with two horizontal and vertical acceleration components. In the above mentioned tables, each of the parameter LHP, LVP, EHP, EVP show the difference of response compared to the first line of each part.

TABLE I
EVALUATION OF YOUNG'S MODULUS OF FOUNDATION IN HORIZONTAL DISPLACEMENT RESPONSE OF THE PINE FLAT DAM CREST POINT UNDER HORIZONTAL AND VERTICAL COMPONENT OF TAFT 1952 EARTHQUAKE.

$E_V\,(mm)$	$L_V\,(mm)$	$E_H\,(mm)$	$L_H\,(mm)$	E_f
2.5	2.6	37.94	37.98	0.2E
4.1	4.4	29.665	29.405	0.5E
7.8	7.8	31.7	31.34	E
12.2	12.5	36.05	37.98	2E
15.6	16.2	36.2	40.22	5E
20.4	20.4	37.23	40.77	Rigid

Where E is the dam young's modulus.

According to the above table, the response of the vertical component of the earthquake will increase with the increasing of the young's modulus, while horizontal component of the earth quake doesn't follow a regular rule.

But the important part is that in all cases with increasing of the elasticity modulus of foundation, the period of the vibration of system will decrease. As we see in the table, in both Eulerian and Lagrangian method, response sensitivity to the foundation young's modulus will be considerable. And both methods have the same response.

TABLE II
EVALUATION OF THE RESERVOIR BOTTOM ENERGY ABSTRACTION IN RESPONSE OF HORIZONTAL DISPLACEMENT OF THE PINE FLAT DAM CREST POINT, UNDER HORIZONTAL AND VERTICAL COMPONENT OF TAFT 1952 EARTHQUAKE.($E_f \rightarrow \infty$)

$E_{VP}\,(\%)$	$E_V\,(mm)$	$L_{VP}\,(\%)$	$L_V\,(mm)$	$E_{HP}\,(\%)$	$E_H\,(mm)$	$L_{HP}\,(\%)$	$L_H\,(mm)$	β
0	21	0	22.1	0	39.41	0	40.5	∞
-18.5	17.1	-18.5	18	-2.81	38.3	-1.2	40	20
-32.1	14.3	-28.4	15.9	-5.53	37.23	-3.23	39.2	9
-55.8	9.3	-53.5	10.3	-14.69	33.62	-11.82	35.7	3

From the results of the Table 2 is understood that the materials of the reservoir's bottom decrease the response under the horizontal component of earthquake about 7 percent and this decrease will go to 38 percent under the vertical component. So we conclude that the reservoir's bottom materials have a great deal with the absorbing of the earthquake energy and decreasing its response in vertical acceleration of the earth. And this effect in both methods is the same.

TABLE III
EVALUATION OF RESERVOIR DEPTH IN HORIZONTAL DISPLACEMENT RESPONSE OF THE PINE FLAT DAM CREST POINT UNDER HORIZONTAL AND VERTICAL COMPONENT OF TAFT 1952 EARTHQUAKE.

$E_{VP}\,(\%)$	$E_V\,(mm)$	$L_{VP}\,(\%)$	$L_V\,(mm)$	$E_{HP}\,(\%)$	$E_H\,(mm)$	$L_{HP}\,(\%)$	$L_H\,(mm)$	Res.depth
0	13.16	0	13.58	0	37.44	0	38.04	full
-79.33	2.72	-71.43	3.88	-40.52	22.27	-41.48	22.26	2/3
-70.29	3.91	-72.09	3.79	-49.39	18.95	-50.42	18.86	1/3
-67.86	4.23	-69.37	4.16	-50.29	18.61	-51.05	18.62	empty

From table 3, it can be understood that with decreasing the depth of the reservoir, under the horizontal component, the response will decrease to 51 percent and under vertical component of the earthquake it will decrease to 80 percent.

Then we concluded that the reservoir depth has great effect on the dam's response. So reservoir interaction with dam and foundation is very important and can't be ignored. . Also the difference between full reservoir and reservoir with $2/3$ depth is a lot but this difference between $2/3$ depth and less is very little or zero.

TABLE IV
EVALUATION OF RESERVOIR BOTTOM SLOPE IN DISPLACEMENT RESPONSE OF THE PINE FLAT DAM CREST POINT UNDER HORIZONTAL AND VERTICAL COMPONENT OF TAFT 1952 EARTHQUAKE.

E_{VP}(%)	E_V (mm)	L_{VP}(%)	L_V (mm)	E_{HP}(%)	E_H (mm)	L_{HP}(%)	L_H (mm)	Res. bottom slope(%)
0	13.5	0	13.5	0	39.96	0	39.83	0
-3.9	13	-0.2	13.48	-0.35	39.82	-0.93	39.46	4
-8.8	12.3	-4.2	12.9	-0.3	39.84	-1.63	39.18	8

This table results indicates that the reservoir bottom slope has little or no effect on the response of system under earthquake and with increasing the slope the response will decrease.

TABLE V
EVALUATION OF EFFECT OF UPSTREAM DAM FACE SLOPE IN DISPLACEMENT RESPONSE OF THE PINE FLAT DAM CREST POINT UNDER HORIZONTAL AND VERTICAL COMPONENT OF TAFT 1952 EARTHQUAKE. (TOTAL HEIGHT)

E_{VP}(%)	E_V (mm)	L_{VP}(%)	L_V (mm)	E_{HP}(%)	E_H (mm)	L_{HP}(%)	L_H (mm)	Upstream face slope (%)
0	13.5	0	13.5	0	39.95	0	39.83	0
8.5	14.7	8.7	14.65	-10.26	35.85	-8.21	36.56	5
-2.3	13.2	-2.3	13.2	-28.34	28.63	-27.32	28.95	10
-18.4	11	-19.3	10.9	-42.1	23.13	-42.53	22.89	15

TABLEV I
EVALUATION OF EFFECT OF UPSTREAM DAM FACE SLOPE IN DISPLACEMENT RESPONSE OF THE PINE FLAT DAM CREST POINT UNDER HORIZONTAL AND VERTICAL COMPONENT OF TAFT 1952 EARTHQUAKE. (HALF- HEIGHT)

E_{VP}(%)	E_V (mm)	L_{VP}(%)	L_V (mm)	E_{HP}(%)	E_H (mm)	L_{HP}(%)	L_H (mm)	Upstream face slope (%)
0	2.7	0	2.7	0	31.94	0	31.53	0
2.5	2.72	2.9	2.8	-2.16	31.25	-2.57	30.72	5
4.2	2.83	5	2.84	-3.82	30.72	-4.13	30.17	10
8	2.9	8.2	2.91	-5.16	30.29	-5.64	29.75	15

It is noticed that in table 5 and 6 with increasing the upstream dam face slope the system response will decrease and this decrease will be more than the time when half part is at slope. The decrease is about 50 percent and isn't negligible.

VI. CONCLUSION

With considering of the analysis and hypotheses in this research we study the advantages and disadvantages of the Eulerian and Lagrangian methods:

1- In analyses of the methods we understand that Lagarngian method has greater answer in compare of response that Eulerian method so in designing it gives stronger caeses.

2- With increasing of young's modulus of foundation, the responses under vertical component of the earthquake will increase, but horizontal component of the earthquake doesn't follow a regular rule. But in all cases with increasing of young's modulus of foundation, the vibration period of system will decrease.

3-Materials 0n the reservoir bottom has great influence in absorbing of earthquake waves and energy and decreases the system response under the vertical component of the earthquake and this effect is also important for horizontal component. This effect in Lagrangian and Eulerian methods is the same.

4- with decreasing the depth of reservoir, in horizontal acceleration of the earth the response will decease to 51 percent and this amount in vertical acceleration is 80 percent. It is concluded that reservoir depth has great effect on the dam- reservoir- foundation system and their interaction isn't negligible.

5- The reservoir bottom slope has little effect on response of the system under the vertical and horizontal acceleration of the earth. With the increase of slope, the response will decrease.

From table 4 we understand that the slope of the bottom of reservoir has little or no effect on the horizontal acceleration of the earth. And with increasing of the slope of the reservoir bottom, the responses under the vertical and horizontal acceleration will decrease.

6- With increasing of the upstream dam face slope, the response of the system will decrease. And this decrease will be more if all part of the dam is at slope than half part of the dam. And this 50 percent can not be ignored.

7- for the better planning and fast analysis, the advantage of the Lagrangian method is that total system have one variable, so the stiffness matrix and mass matrix will have diametrical shape, while in the Eulerian method because of the difference of the structure and water variable, the equations are different, and analyses of the pine flat dam with Lagrangian method needs more time.

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