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Reducing the Impact of Uplift Pressures on the Base of a Concrete Dam by Configuration of Drainage Holes (Hypothetical Case Study)

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Abstract:

A study of the impact of the uplift pressures upon base of a typical water retaining structure was presented. This work was conducted through numerical analysis by finite element method to evaluate the hydraulic uplift pressure distribution generated by the calculated flow. Also, the effects of position of either single or dual drainage holes on uplift pressures characteristic were included. A hypothetical case was solved for three types of drainage holes, and the reduction of uplift pressures were computed in each case, a comparison was presented and the results showed that it can produce the desired reduction in hydraulic uplift pressures by using two drainage holes at equidistance of 8m from upstream edge of structure floor.

1-Introduction

Uplift and pore water pressures are the main hydraulic factors to be considered in the design of hydraulic structures. In particular, pore water pressure, which is the penetrating pressure, may accelerate structural corrosion and consequently increase seepage. Avoiding high water pressure is, therefore, one of the main concerns in the design of underwater structures. Reduction of pore water pressure is often necessary to secure structural safety, and is generally achieved by adopting minor filter drainage systems. In addition, it is not possible to install such a drainage systems for various types of hydraulic structures such as single sell tunnel. As an alternative measure, drainage holes system has been increasingly used (Shin et al., 2009).

In this work, the hydraulic behavior of a drainage-holes drain are investigated using the numerical method, and the applicability of the drainage-holes system are evaluated by performing a numerical hypothetical study for various design parameters. Based on the analysis results, some design conclusions are made.

2- Review of Literatures

Da Silva (2005) investigated the influence on the uplift pressures of the number and position of the drainage galleries, of the diameter, spacing, length, roughness and inclination of drains for a dam on homogeneous and isotropic material.

The analyses were performed using a nonlinear tri-dimensional finite element model. It was concluded that the drain length causes the greatest reductions in the uplift pressures, followed by its spacing and diameter. Tilting the line of drains offers no advantages except when using two lines of drains. The roughness of the drains normally used in concrete gravity dams causes little impact on the uplift pressures. The anisotropy of the foundation materials has great influence on the uplift pressures.

Y. Chen et al. (2008) presented a study to accurately characterize the boundary conditions of the drainage systems, to reduce the difficulty in mesh generation resulting from the drainage holes with small radius and dense spacing, and to eliminate the singularity at the seepage points and the resultant mesh dependency. Numerical stability and robustness of the proposed method were guaranteed by an adaptive procedure for progressively relaxing the penalized Heaviside function associated with the formulation of the discrete variational inequality.

Yu et al. (2009) analyzed the effect of the parameter of concrete cut-off wall, the permeability coefficient of cut-off wall, drainage hole and grout curtains on seepage flow of the Fengman dam. It is concluded that the grout curtain, which was performed during the dam construction, is not effective and the leakage occurs under the main grout curtain. For that reason, a cut-off wall was recommended.

Chen et al.,(2010) used composite element method (CEM) to formulated the seepage analysis of rock masses containing fractures and drainage holes. Each fracture or drainage segment was treated as a special sub-element having definite seepage characteristics, and was located explicitly within the composite element, the validity and reliability of the CEM was verified by a numerical example. The application and comparative study was presented for the Baozhusi dam foundation (gravity dam on the Bailong River in China).

Shamsai et al., (2010) discussed a mathematical model of moving water under concrete dam in porous media. In this case, according to discretization methods of the governing equations in porous media such as finite difference and finite element, finite volume techniques, the last one was selected. For solving the equations, unstructured mesh was used. The effect of under relaxation coefficient to increase the rate of convergence was illustrated. This factor was calculated and found equal to 0.995 that showed 10 percent improvement in rate of convergence. The results of seepage discharges by using three powerful seepage codes (Seep/w, Mseep and Plaxis) that are based on finite element method were compared with results of FVseep model which was based on finite volume method and showed 5.1 percent improvement in accuracy.

Azizi et al., (2012) studied the Yusufkand Mahabad diversion dam to determine the effect of weep holes location and different depth of the dam cutoff walls on uplift pressure and on exit hydraulic gradient. Results showed that upstream cutoff with 8 meter depth decreases uplift force about 63% and decreases exit gradient 79% respect to without cutoff case. Installing weep hole in downstream stilling basin decreases uplift force 8% and decreases exit gradient 74% more than without weep hole.

Mansuri and Salmasi (2013) studied the effectiveness of using horizontal drain and cutoff wall in reducing seepage flow from an assumed heterogeneous earth dam. Various horizontal drain lengths and cutoff wall depth were examined under the earth dam in different location of foundation; the results showed that increasing horizontal drain length, cause slightly in increasing seepage rate and increasing hydraulic gradient. Optimum location of cut off wall for reduction of seepage rate and piping was in the middle of dam foundation. By increasing in cut off wall depth, seepage from earth dam and its foundation was reducing. Different location of cut off wall in dam foundation had little effect on exit hydraulic gradient and always it was less than unity. Installation of cut off wall in middle of foundation, results 19.68 percent decreasing in hydraulic gradient respect to existent of cut off wall in the upstream of dam.

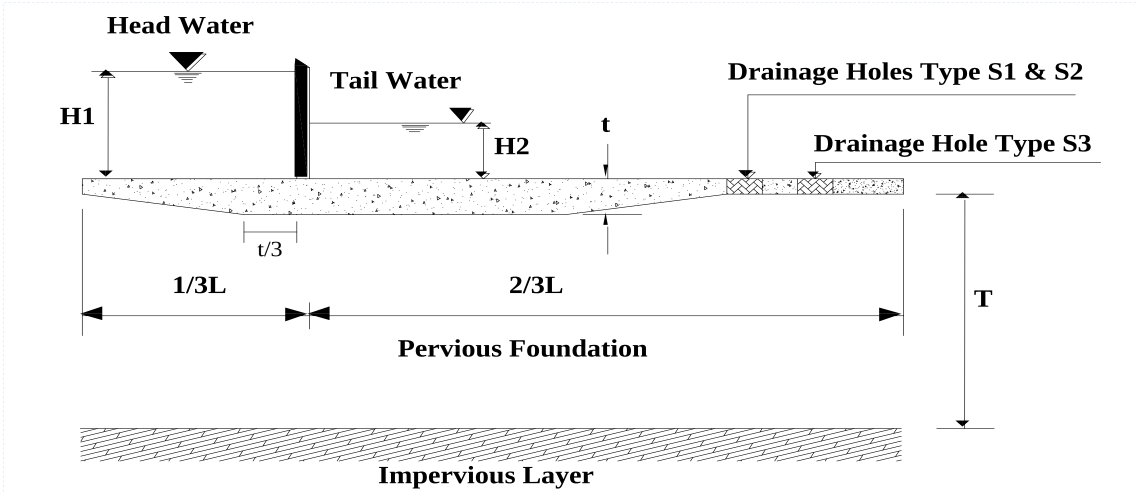
3- Governing Equation

The groundwater flow equation for two dimensional of incompressible flows in porous media can be expressed as (Manna et al., 2003):

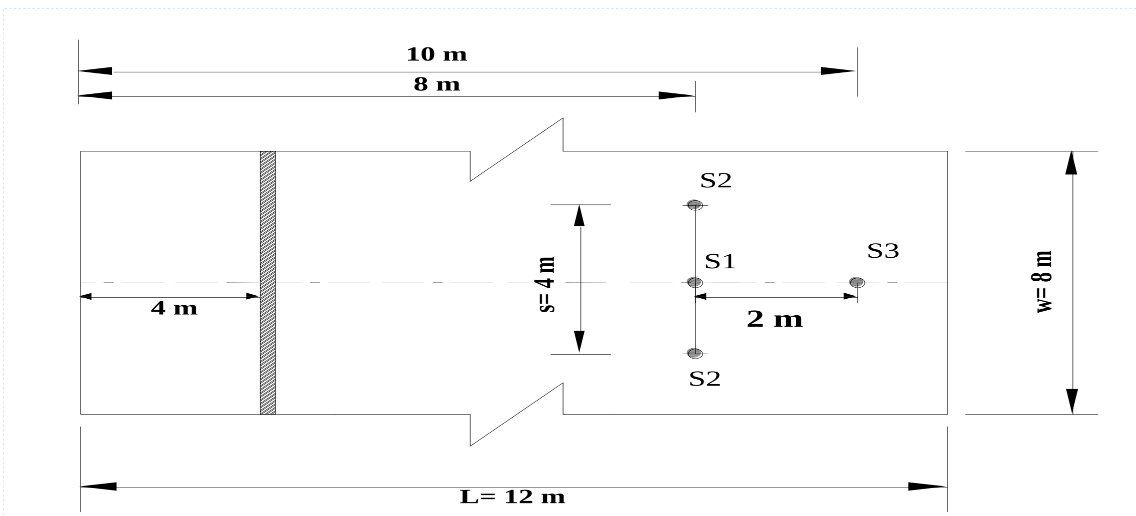
$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)+Q+C_s \frac{\partial h}{\partial t}=0 \qquad (1)$$

Where, h is the pressure head, Kx and Ky represents the hydraulic conductivities in X and Y directions, Q a specified inflow or outflow, Cs is the specific storage, x and y are rectangular coordinates, and t is the time. A definition sketch to demonstrate this work is shown in figure (1). For two dimensional steady state flow and setting Q and Cs equal to zero, equation(1) reduced to the form:

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)=0 \qquad (2)$$



(A): Typical cross section of hydraulic structure



(B): Plan view of dam-structure

Figure (1): Definition sketch for hypothetical case study

4- Boundary conditions

The following boundary conditions, shown in figure (2), were used in this work:

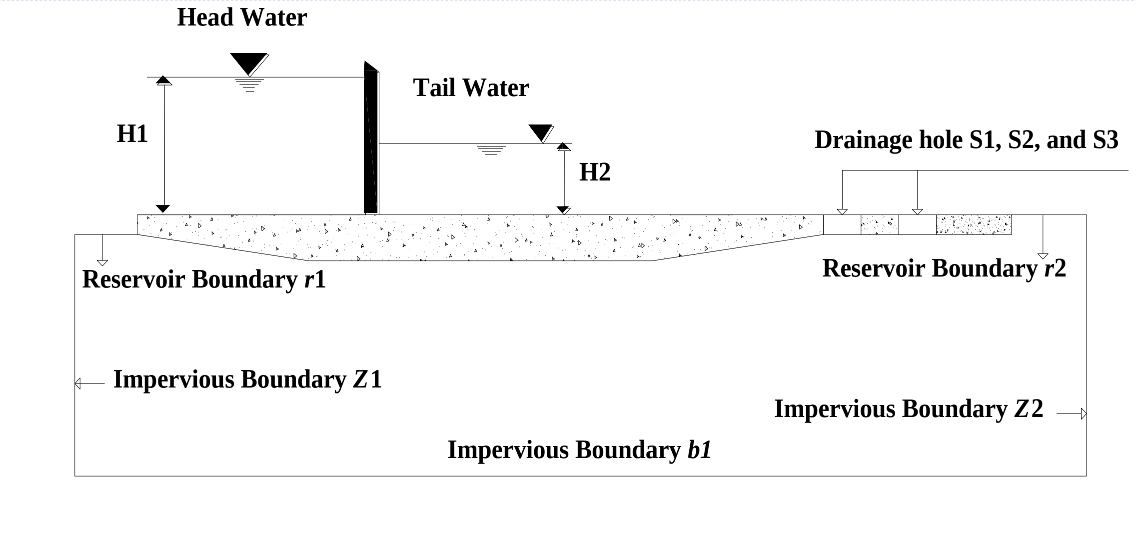


Figure (2): Boundary conditions

a- reservoir boundaries r1, and r2: since the water depth above such boundaries are known (H1), and (H2) respectively; so, the pressure head distribution on these boundaries would be constant i.e., equipotential lines, and equal to:

$$h = \frac{\psi p}{\gamma_w} + y \qquad (3)$$

b- impervious boundaries Z1, Z2, and b1: in which the velocity component normal to the boundary is equal to zero, i.e. these boundaries represents stream lines of constant stream function, that is:

$$\frac{\partial h}{\partial n} = 0 \qquad (4)$$

c- seepage surface on drainage holes S1, S2, and S3: the pressure head in drainage holes will be equal to the pressure head in the downstream, i.e. different ratio of pressure head($\overline{H}$) equal to zero, that is:

$$\overline{H} = \frac{h - H2}{H1 - H2} \qquad (5)$$

5- Numerical Formulation for Finite Element Method

A spatial discretization of equation (2) within pervious soil foundation, figure (1-a) was applied with a Galerkin type finite element method (Rao, 2004). The unknown variable h through the solution domain is approximated by h^e:

$$h^e(x, y) = \sum_{i=1}^{N} N_i(x, y) h_i \qquad (6)$$

Where, Ni is the shape function associated with node i, hi is the unknown potential at node i, and N is total number of nodes. The approximate solution for pressure head variation, h(x,y), over the whole domain is given as follows:

$$h(x,y)=\sum_{e=1}^{n_e} h^e=\sum_{e=1}^{n_e}\sum_{i=1}^{N} N_i(x,y)h_i \tag{7}$$

in which ne is the total number of elements in the flow region. For a homogenous, porous media with horizontal impervious bed, equation (2) becomes:

$$K_x\frac{\partial^2 h}{\partial x^2}+K_y\frac{\partial^2 h}{\partial y^2}=0 \tag{8}$$

Substituting equation (7) into equation (8) yields:

$$\left[K_x\frac{\partial^2}{\partial x^2}\sum_{i=1}^{N} N_i h_i\right]+\left[K_y\frac{\partial^2}{\partial y^2}\sum_{i=1}^{N} N_i h_i\right]=R^e\neq 0 \tag{9}$$

In which Re is the elemental residual.

By multiplying equation (9) with weighted functions Wi=1 to N and using Green's theorem, the formulation of seepage phenomenon by using finite element method ended with number of linear equations. These equations can be established in matrix form as:

$$[C]_{N\times N}\{h\}_{N\times 1}=0 \tag{10}$$

in which , [C] is a stiffness/conductance square matrix with N rows equal to number of nodes discretize the domain study, where:

$$[C]=\sum[C^e] \tag{11}$$

Where [C^e] represent element matrix:

$$[C^e]=\int_{\Omega^e}[B^e]^T[D^e][B^e]dxdy=0 \tag{12}$$

$$[B^e]=\begin{bmatrix}\frac{\partial N^e{}_1}{\partial x} & \frac{\partial N^e{}_2}{\partial x} & \cdots & \frac{\partial N^e{}_n}{\partial x}\\ \frac{\partial N^e{}_1}{\partial y} & \frac{\partial N^e{}_2}{\partial y} & \cdots & \frac{\partial N^e{}_n}{\partial y}\end{bmatrix}$$

$$[D^e]=\begin{bmatrix}K_x & 0\\ 0 & K_y\end{bmatrix}$$

{h}N ×1 is unknown vector with N rows, e is element number, Ne shape function of element e, n number of elements, and Ωe domain of element e, Kx and Ky are constant hydraulic conductivities in x and y directions within element e. By solving the preceding system of equations, equation (10), the unknown pressure head hi=1 to N at different nodes can be calculated.

An appropriate mesh in numerical methods is a mesh that can increase the rate of solutions in calculation's procedure and causes the best accuracy with minimum iterations. So, type and the geometry of meshes have a high effect on results in numerical methods. In this work, a special mesh was used that is adapted with our geometry. Nodes are added or removed from the mesh to ensure the required element size distribution is approximated. Two dimensional simulation of homogeneous dam foundation have 105 elements that intersect the media into smaller pieces. Homogeneous foundation of concrete dam model has 12m length and 4 m depth. The simulation showed that the value of seepage discharge has a little variation with longer and deeper models. This is achieved by several running of models.. The value of hydraulic conductivity for soil medium of (0.00001 m/sec.) has been chosen based on literature.

The model primarily computes the spatial distribution of the hydraulic pressure heads in foundation soil, the value of exit gradient at the exit node computed and compared with the safe exit gradient value on the exit node. If the difference between them is greater than a set tolerance number, then iterations have to be repeated. If the difference is smaller than the tolerance, than the solution reaches the final iteration, and the solution obtained is the final solution. Figure (3) shows the flow chart describing the numerical simulation.
A group of experiments were performed by Abd El-Razek and Abo Elela (2002) to determine the optimum location of drainage gallery underneath the gravity dam which was found in the middle of dam floor (b/B =0.5). Stefano et al., (2008) study the influences of the hydraulic uplift pressures underneath the base of a typical concrete gravity dam on its stability, they showed that for design purposes it appears that availability of reliable data on the hydraulic permeability of rock foundations and a computationally advanced distinct element modeling might lead to the acceptance of loads significantly higher than the more conservative estimations obtained from equilibrium analyses. Relief drainage hole efficiency is a function of drain geometry, i.e. diameter, spacing and distance to the upstream face (Novak et al., 2007).

In the this research work, four parameters of the drainage holes are studied numerically to determine their effect on hydraulic uplift pressures acting on the floor of the dam, these parameters are the diameter of the drainage hole (d), distance from the upstream edge of the floor (x), the spacing of the drainage hole in the transverse direction of dam center line (s), and relative water head (H2/H1).

7- Results and Discussion

To study the effect of drainage holes parameters on the percentage of reduction of hydraulic uplift pressures, the following procedure is carried out:

1- For a constant head water level (H1=5m, H2=0), five different values of drainage hole diameters are tested [d =15, 20, 25, 30, 35 cm] respectively, three types of drainage holes are presented in order to study the effect of longitudinal distance (x) and transverses spacing (s) of drainage holes:

a- Drainage hole S1: single drainage hole on center line (s=0), at distance (x=8 m) from the upstream edge of the floor.

b- Drainage hole S2: double drainage hole (s =4m), at distance (x=8 m) from the upstream edge of floor.

c- Drainage hole S3: double drainage hole (s =0), at distance (x= 8m, 10m) respectively from the upstream edge of floor.

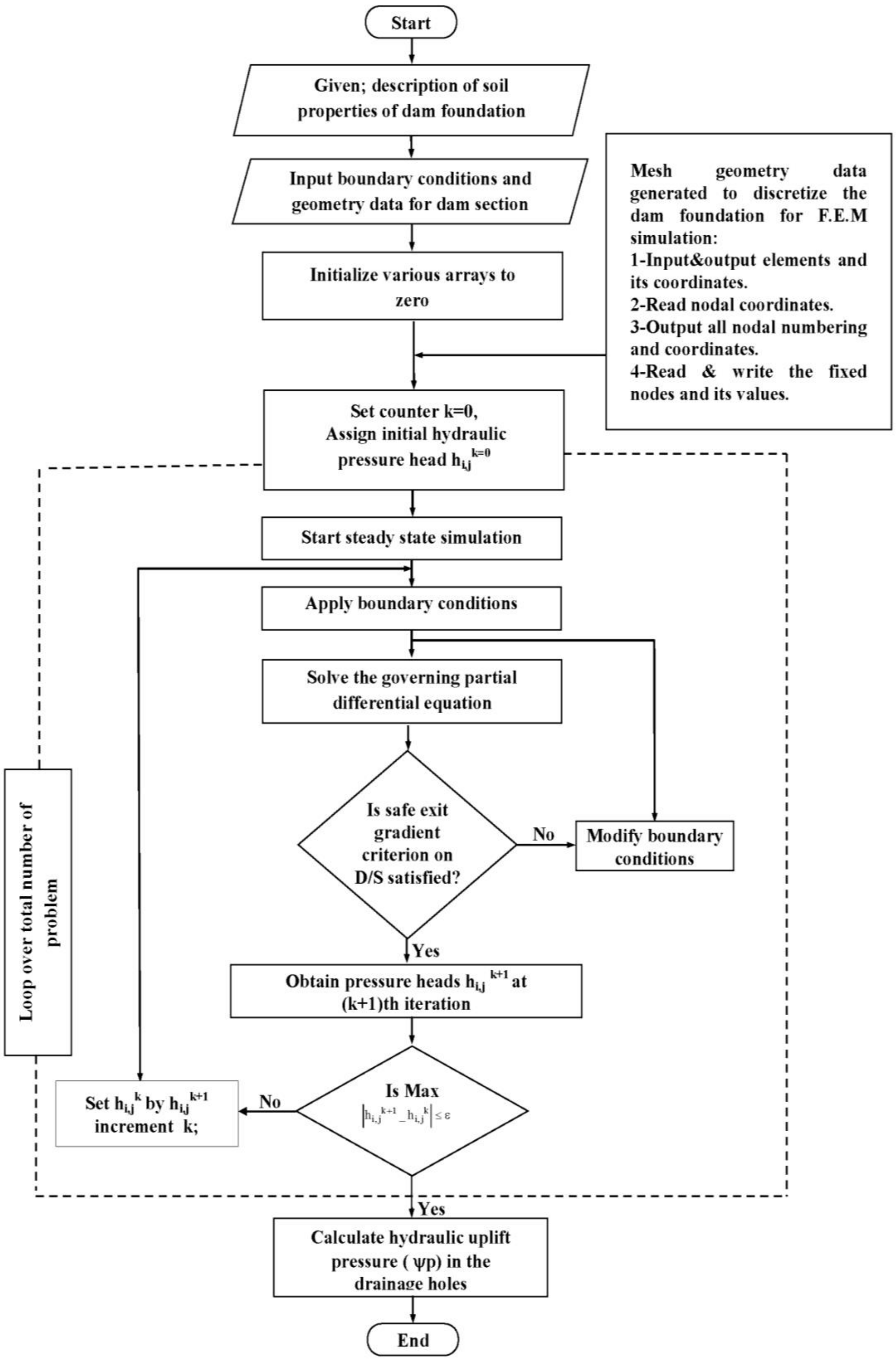


Figure (3): Flow chart of the general numerical model

For each type of control devices for safe hydraulic uplift pressure, to be able to evaluate the performance of such device the most commonly used error measures were computed as summarized by the absolute relative error (APE) as defined below:

$$ARE = \frac{\sum_{i=1}^{N_r} \left| \psi_{p_\circ} - \psi_{p_1} \right|}{\psi_{Po}} \times 100 \qquad (13)$$

And the root means square error (RMSE), which can be defined as (Ersayin, 2006):

$$RMSE = \sqrt{\frac{\sum_{i=1}^{N_r} (\psi_{p_\circ} - \psi_{p_1})^2}{N_r}} \qquad (14)$$

In which;
ψp0 = hydraulic uplift pressure without seepage control device.
ψp1 = uplift pressure head coupled to use of seepage control device.
Nr = number of observations.
The results represent the rate of uplift pressures reduction due to construct of different types of drainage holes (S1, S2, and S3) versus the effect of relative water head (H2/H1) for three different values (0,0.45and 0.60) as shown in tables (1, 2, and 3).

Table (1): Reduction of hydraulic uplift pressures for $(\frac{H_2}{H_1} = 0)$, (H_1=5m).

Type of drainage holes	Diameter of drainage holes (m)	Uplift Reduction Rate %	ARE%	RMSE
S1: Single drainage hole(x=8m, s=0)	d = 0.15 d=0.20 d=0.25 d=0.30 d=0.35	9.4 16.3 32 72.7 140.2	2.819 4.889 9.598 21.805 42.051	0.726
S2: Double drainage hole(x=8 m both, s=4m)	d =0.15 d=0.20 d=0.25 d=0.30 d=0.35	70.5 116.3 152.0 182.7 205.9	21.145 34.883 45.591 54.990 61.757	1.532
S3: Double drainage hole(x=8m, 10m, s=0 m)	d =0.15 d=0.20 d=0.25 d=0.30 d=0.35	116.7 177.0 231.7 271.9 319.3	1.167 1.77 2.317 2.719 3.193	2.343

Table (2): Reduction of hydraulic uplift pressures for ($\frac{H_2}{H_1} = 0.45$), (H_1=5m, H_2=2.25m)

Type of drainage holes	Diameter of drainage holes (m)	Uplift Reduction Rate %	ARE%	RMSE
S1: Single drainage hole(x=8m, s=0)	d = 0.15	130.9	32.059	1.628
	d=0.20	144.0	35.268	
	d=0.25	173.8	42.566	
	d=0.30	178.5	43.717	
	d=0.35	180.7	44.256	
S2: Double drainage hole(x=8 m both, s=4m)	d =0.15	238.1	41.685	2.236
	d=0.20	212.0	48.077	
	d=0.25	185.7	54.518	
	d=0.30	159.5	60.936	
	d=0.35	141.7	65.295	
S3: Double drainage hole(x=8m, 10m, s=0 m)	d =0.15	167.8	38.914	1.90
	d=0.20	177.6	41.187	
	d=0.25	195.8	45.408	
	d=0.30	201.4	46.706	
	d=0.35	205.1	47.565	

Table (3): Reduction of hydraulic uplift pressures for ($\frac{H_2}{H_1} = 0.6$), (H_1=7.5m, H_2=4.5m).

Type of drainage holes	Diameter of drainage holes (m)	Uplift Reduction Rate %	ARE%	RMSE
S1: Single drainage hole(x=8m, s=0)	d = 0.15	079.3	12.323	1.446
	d=0.20	131.5	20.435	
	d=0.25	141.1	21.927	
	d=0.30	160.5	24.941	
	d=0.35	187.8	29.184	
S2: Double drainage hole(x=8 m both, s=4m)	d =0.15	164.9	25.625	2.876
	d=0.20	222.1	34.514	
	d=0.25	279.2	43.387	
	d=0.30	336.4	52.276	
	d=0.35	382.1	59.378	
S3: Double drainage hole(x=8m, 10m, s=0 m)	d =0.15	118.0	17.763	1.687
	d=0.20	172.6	25.982	
	d=0.25	188.1	28.315	
	d=0.30	169.3	25.485	
	d=0.35	185.8	27.970	

A comparison among the calculated root mean square error measures for each type of drainage hole versus different relative water head (H2/H1) are shown in figure(4).

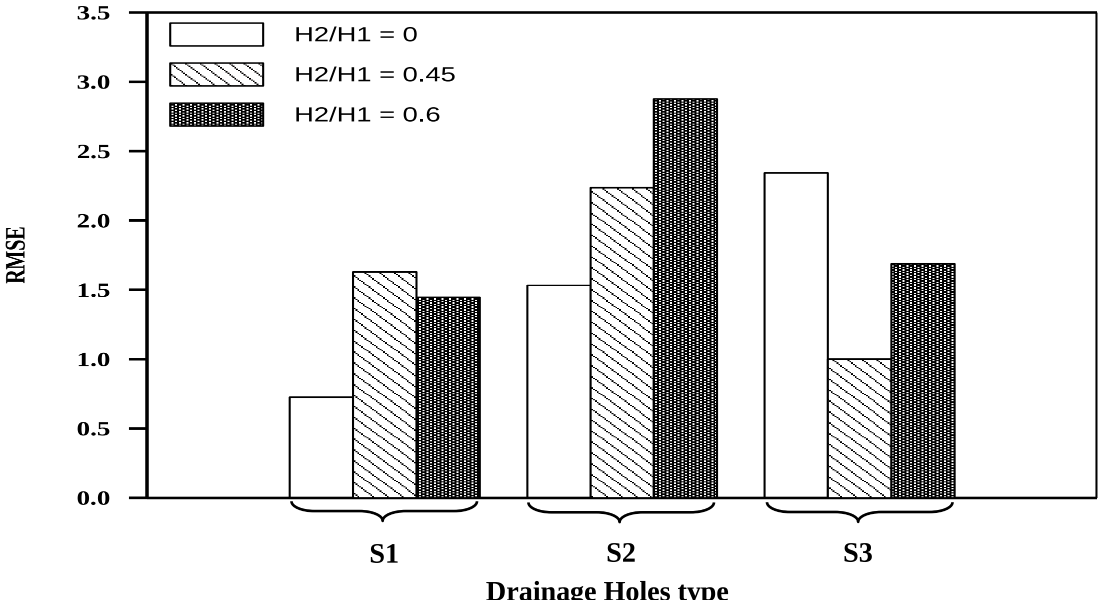


Figure (4): Comparison of measured root mean square errors versus relative water head for different types of drainage holes

The effect of longitudinal distance from the upstream edge of the structure floor , and transverse spacing are investigated in figure(4), the results show that for a double drainage holes and both at 8m from the upstream edge, the root mean square error is (2.876) for a case of relative water head ratio of(0.6). The corresponding rate of the average reduction of hydraulic uplift pressure head is (2.769m). The effect of diameter of drainage holes on the uplift pressure values for different relative water heads are shown in figures (5, 6, and 7).

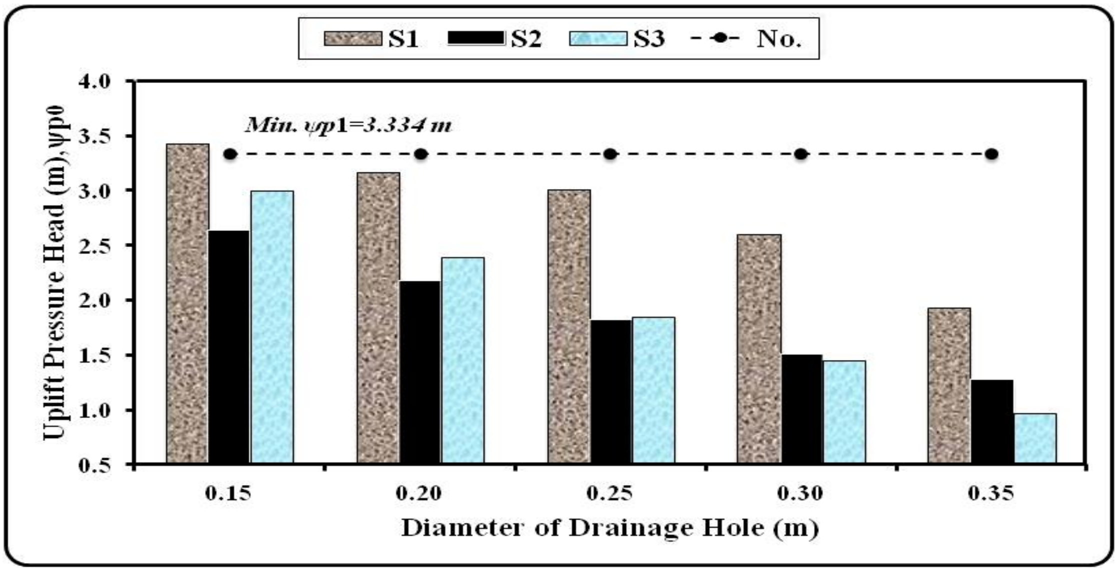


Figure (5): Reduction in uplift pressure head (ψp_0) versus drainage hole diameters for different drainage hole types

$$\left(\frac{H_2}{H_1} = 0\right)$$

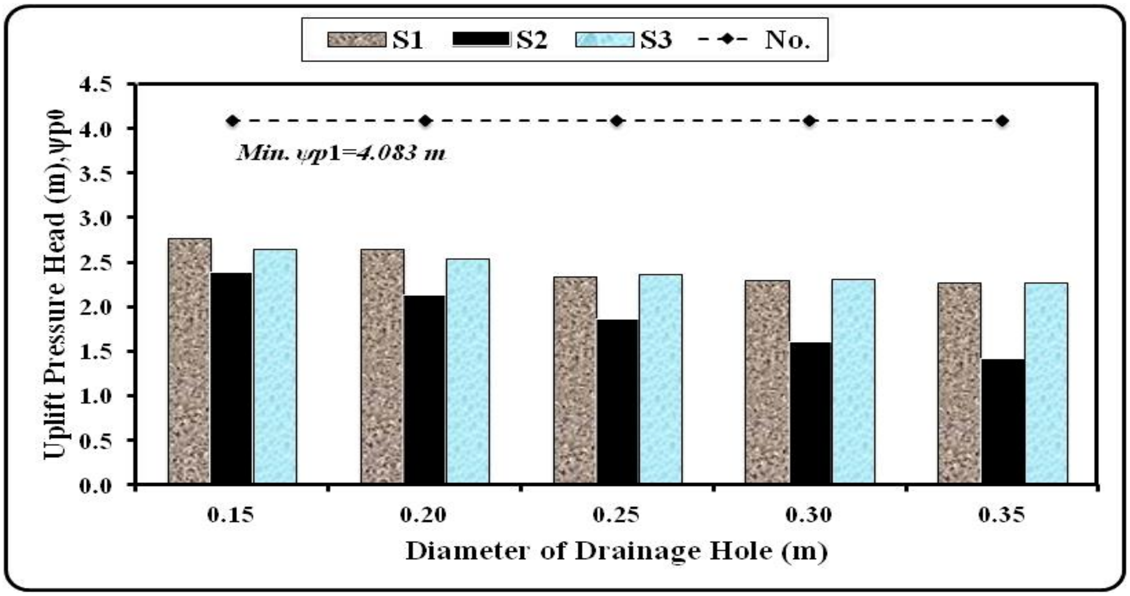


Figure (6): Reduction in uplift pressure head (ψp_0) versus drainage hole diameters for different drainage hole types

$$\left(\frac{H_2}{H_1} = 0.45\right)$$

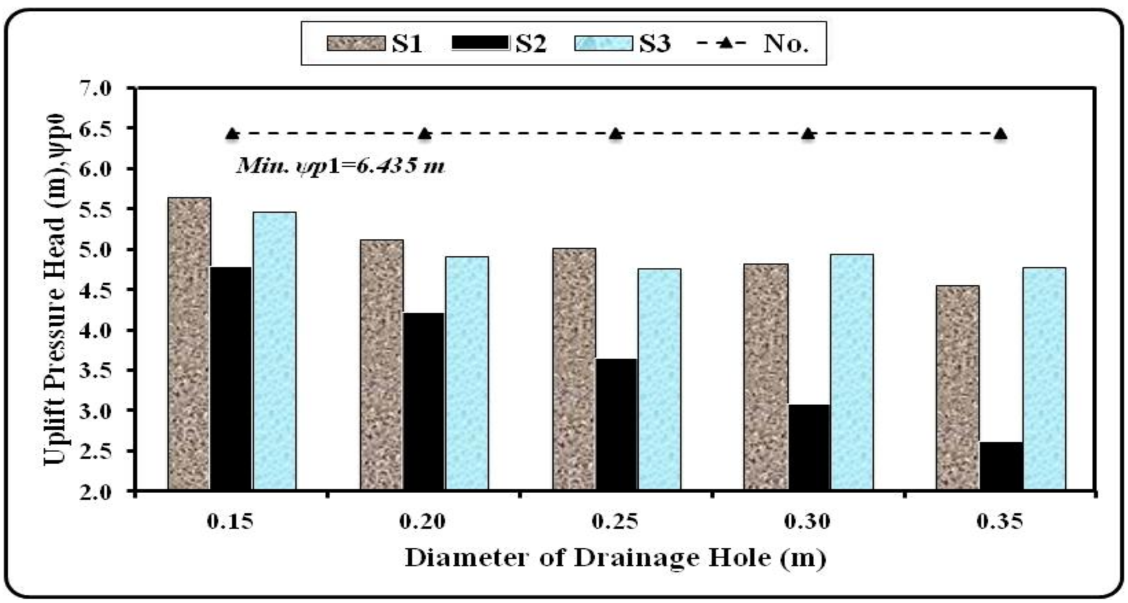


Figure (7): Reduction in uplift pressure head (ψp_0) versus drainage hole diameters for different drainage hole types $\left(\frac{H_2}{H_1} = 0.6\right)$

The value of reduction of uplift pressure increase as the area of drainage hole increased as shown in figures (5, 6, and 7), for a relative water head ratio (H2/H1=0) the average reduction in uplift pressure heads is increased rapidly as drainage hole diameter increased, and the average percentage of pressure reduction is equal 10.8 % as observed in figure (5). The effect of longitudinal spacing of drainage hole on reduction of uplift pressure heads are indicated in figures (6, 7), for double drainage holes of equidistance from upstream floor edge of 8m the average reductions are (2.208m, 2.77m) for relative water head ratio of (0.45, 0.6) respectively, it is seen that the effect of change of relative head values from (2.75m to 3.0m) involve decreasing of uplift reduction by (0.69m). The transverse spacing of drainage holes on the reduction of uplift heads effects on the shape of uplift pressure diagram.

8- Conclusions

According to results obtained of this work, it can be concluded the following points:

1- For a longitudinal distance of 8m from the upstream edge of structure floor, use double drainage holes located on the dam base centerline, the average reduction of uplift pressure head values from 1.40m to 2.77m.

2- Increasing the drainage hole diameter from (o.15m to 0.35m) of two drainage holes of x=8m , yields maximum reduction in uplift pressure heads from 1.345m to 2.172m for a range of relative water head from 0 to 0.6.

3- The effect of transverse spacing is small and effects on the shape of uplift pressure diagram.

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