The study of Piping in some hydraulic structures in Urmia region, Iran, using the PIPING model

Étude de la canalisations dans des structures hydrauliques de la région d'Urmia, Iran, en utilisant le modèle PIPING

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ABSTRACT: Six diversion dams in Shar-Chai, Nazlu-Chai, and Baranduz-Chai rivers in Urmia plain, Urmia, Iran, were selected to investigate their safety against piping. The PIPING code based on the Sellmeijer's design formula and the probabilistic method was used as the analytical model. Some sensitivity analysis was done using the collected prototype and laboratory data to investigate the model's compatibility and accuracy. The model produced reasonable results at the end of this analysis. The selected diversion dams were then analyzed for their probability of failure due to piping. The results were compared with the predetermined target probabilities of failure and showed safe configurations for most of the selected dams.

The partial safety factors for the Sellmeijer's design formula were calculated to be used when designing water retaining structures for piping in Urmia region.

The comparison of the Sellmeijer's method with the empirical methods of Bligh and Lane showed that the Sellmeijer's method requires shorter seepage length and produces economical design for water retaining structures against piping.

RÉSUMÉ: Six barrages de déviations dans les fleuves de Shar-Chai, de Nazlu-Chai, et de Baranduz-Chai dans la plaine d'Urmia, Urmia, Iran, ont été choisis pour étudier leur sécurité contre la canalisations. Le code PIPING basé sur la formule de conception de Sellmeijer et la méthode probabilistique a été utilisé comme le modèle analytique. Quelque analyses de sensibilités ont été faites en utilisant le prototype et les données rassemblés de laboratoire pour étudier la compatibilité et l'exactitude de ce modèle.
Le modèle produie des résultats raisonnables à la fin de cette analyse. Les barrages de déviations choisis ont été alors analysés pour leur probabilité de rupture due à la canalisations. Les résultats ont été comparés aux probabilités du critère de la rupture prédéterminé et ont montré des configurations solides pour plupart des barrages choisis.

Les coefficients de sécurités partiels pour la formule de conception de Sellmeijer ont été calculés afin d'être utilisés pour la conception de canalisations des structures de retenue de l'eau dans la région d'Urmia.

La comparaison de la méthode de Sellmeijer avec les méthodes empiriques de Bligh et de Lane a prouvé que la méthode de Sellmeijer exige une longueur plus courte d'infiltration et en plus la conception économique pour les structures de retenue de l'eau contre la canalisation.

1 INTRODUCTION

Piping is a form of seepage erosion, the general name for the adverse effects of groundwater flow on soil stability. High seepage pressure may remove soil material to such an extent that geotechnical structures may, and do, collapse. The actual word "piping" refers to the development of channels, which form at the downstream side of the structure where the flow lines converge and high seepage pressures occur. The erosion process then begins and develops backward. If this process continues, the eroded channels may reach upstream and cause the total collapse of the structure.

For safety purposes, the water retaining structures such as embankment dams for flood control and river dams for water flow control, should be designed against piping. The whole mechanism is complex and to describe it mathematically, use is made of branches of soil mechanics, hydraulics, and groundwater flow.

2 EMPIRICAL AND ANALYTICAL MODELS FOR PIPING

2.1 *Empirical models*

In the context of weir and dam design on permeable foundations, seepage erosion through the foundation plays an important role and should be evaluated for safe design. As a result of research, an empirical rule relates the hydraulic head across the structure H, to the length of seepage, L. For simple structures, L may be assumed to be equal to the length of the weir or dam. The rule reads as follows:

$$L = C\,H \tag{1}$$

Where C is a coefficient that depends on the geometry and soil parameters. For the first time this relation appears in the work of Clibborn and Beresford (Clibborn & Beresford 1902). Bligh used the same concept and proposed his "Line of creep method" (Bligh 1910). In this method, he assumed that the water follows a path along the line of contact of the dam foundation with the foundation material which he called it the line of creep. He suggested some values for the coefficient C that could only be obtained experimentally.

Later in 1935, Lane introduced a new method for analyzing the piping phenomena. After studying more than 200 masonry dams with different foundation materials that experienced failure due to piping, he suggested his "weighted creep method" (Lane 1935). According to his method, a weight of one third is given to the horizontal or slightly inclined path as compared with the other sections of the path. His rule reads as follows:

$$L_{we} = L_h / 3 + L_v = C_{we}\,H \tag{2}$$

Where L_{we} is the weighted seepage length, L_h is the horizontal or near horizontal path, L_v is the vertical or near vertical path, and C_{we} is the weighted creep ratio. According to his studies on more than 200 dams, he suggested some empirical values for C_{we} for different soil materials. The advantageous of Lane's method to that of Bligh is that it allows for shorter seepage length than that of Bligh and also allows the evaluation of the piping for structures having vertical or near vertical elements such as sheet piles and cutoff walls.

2.2 *Analytical models*

After Lane up to the late 1970's research had been concentrated on the heel sheeting type of structures but piping had been mentioned in only a few articles. A research program was started in 1972 in the Netherlands to develop a less conservative rule than the rules of Bligh or Lane. A series of small scale laboratory tests were performed from 1973 until 1983. The typical width of the model dikes was 80 centimeters. Examples are the tests performed in Germany and in the Netherlands. Miesel (1978), Muller-Kirchenbauer (1978), and Hannes et al. (1985) performed model tests on piping from holes made at the top of a confined layer. De Wit et al. (1981) reported experiments in partly covered sand layers.

Theoretical methods that have been contributed to the understanding of the piping have been reported (Sellmeijer 1988, Sellmeijer & Koenders 1991). Muller-Kirchenbauer (1978) and Sellmeijer (1981) used solutions for steady flow and applied conformal mapping. Hannes et al. (1985) worked with a numerical method. In the Sellmeijer's theoretical method, the erosion mechanism plays an important role in the analysis and will be described in the following paragraph.

2.2.1 *Sellmeijer's analytical model*

The study reported by Sellmeijer contains some visual tests along with the analytical study on piping. He observed that an increase in the hydraulic head caused some sand transport at the downstream side of the structure creating a sandboil and a slit. This resulted in a new steady state situation which could be defined by the limit equilibrium of the sand (Sellmeijer 1988, Weijers & Sellmeijer 1993). The seepage flow (or groundwater flow) and the limit equilibrium of the sand in the sand boil and the slit, are two main features of the piping phenomena in this study.

The Sellmeijer's analytical study resulted in a new design formula for piping which reads as follows (Sellmeijer 1988):

$$H(\tfrac{1}{2}L) = C(\tfrac{1}{2}L)\, \gamma'_p/\gamma_w \tan\theta\,(0.674 - 0.102\, LnC)\, L$$

$$(D/L)^{[0.28\,/\,\{(D/L)2.8-1\}]} \tag{3}$$

$$C(\tfrac{1}{2}L) = (1/\hat{C})\{[2\,(D_{70}/D_{10})^2\, D_{70}\,/\,L]/(k\,/\,D_{10}^{\,2}\,)\}^{\frac{1}{3}} \tag{4}$$

Where $H(\frac{1}{2}L)$ is the hydraulic head across the structure (m), C is the fabric factor, L is the length of the structure (or the seepage length) (m), D is the depth of the aquifer (m), D_{70} is the grain diameter at 70% passing by weight (m), D_{10} is the grain diameter at 10% passing by weight (m), k is the intrinsic permeability of the soil (m^2), $\hat{C}$ is the drag factor ($\hat{C} = 4/\pi\eta$, where η is the White factor which ranges between 1/4 to 1/3, (White 1940)), γ'_p is the submerged unit weight of the soil particles (kN/m^3), γ_w is the unit weight of water (kN/m^3), and θ is the bedding angle of the sand (degree). The term "$\frac{1}{2}L$" in the parentheses in Equations 3 and 4 indicates that the critical head level H is reached when the length of the eroded slit underneath the structure is one half of the length of the structure (or the seepage length).

Equation 4 uses the intrinsic permeability (k) of the foundation soil. Since measurement of intrinsic permeability has some practical difficulties, the use of permeability K instead of intrinsic permeability is desirable in practical point of view. Considering the relationship of $K=kg/v$ between these permeabilities, and assuming the value of $v = 1.308x10^{-6}$ for kinematic viscosity at 10 °C which is the average temperature for soil water in the foundation material in Urmia region, by rearrangement, Equation 4 can be rewritten as follows:

$$C(\tfrac{1}{2}L) = 193.7\eta\,(D_{70}^{\,3}\,/KL)^{\frac{1}{3}} \tag{5}$$

Now using Equations 3 and 5 is more practical than Equations 3 and 4 (Calle & Badv personal communications).

When one uses Sellmeijer's design formula (Equations 3 and 5) in practical situations, two difficulties may arise. First, some of the parameters which may strongly influence the results are difficult to estimate, resulting in substantial uncertainty. Secondly, due to lack of long term experience, little is known about the safety factor requirements that should be taken into account when using the computational model. It was expected that the probabilistic analysis would be helpful to investigate the model's sensitivities and to assess safety factor requirements for routine design. Moreover, such an approach would facilitate the comparison of designs based on this new model, with designs based on the classical empirical rules.

2.2.2 *Probabilistic analysis and the "PIPING" code*

According to the Sellmeijer's computational model, failure due to piping will occur when the maximum difference of potential head across the structure (Δh), which occurs during a flood with a specific return period, exceeds the limit state difference of head (ΔH), as determined from Equations 3 and 5. Thus, piping will occur when:

$$Z=\Delta H\,(\theta, K, D_{70}, \eta, L)-\Delta \mathrm{h(Q)} < 0 \tag{6}$$

The function Z, which is random, is referred to as reliability function; $Z>0$ indicates a safe region in the parameter space, $Z<0$ an unsafe region, and $Z=0$ the limit state function. The probability of failure equals the amount of joint probability density contained in the unsafe region (Calle et al. 1989). The target probability of failure due to piping should be assessed consistently with the overall safety of the structure. In the case of water retaining structures, various mechanisms of failure are considered. Examples are overtopping due to high river head or breach of the structure due to geotechnical instability of the slopes. The target probability of failure due to piping is assessed at two percent of the overtopping risk (Calle et al. 1989).

As indicated, due to some uncertainties associated with the assessment of the soil parameters in the Sellmeijer's formula, the probabilistic analysis is found to be the appropriate method to consider these uncertainties in the design. The Sellmeijer's formula along with the probabilistic method of analysis was implemented in a computer model "PIPING".

The probabilistic FORM package developed by the Technical University of Muenich (Fiessler 1979) has been implemented in the PIPING computer code. This code was developed at the Ministry of Transport, Public Works and Water Management, Road and Hydraulic Engineering Division of the Netherlands with the cooperation of the Delft Geotechnique and Technical University of Delft.

The PIPING program requires the following input data in the calculations:

The mean and standard deviations of the parameters: (1) seepage length (L) (m), (2) depth of aquifer beneath the structure (D) (m), (3) the bedding angle of the soil particles (θ) (degree), (4) soil permeability (K) (m/s), (5) the grain diameter at 70% passing by weight (D_{70}) (m). It also requires the submerged unit weight of soil particles, γ'_p (kN/m^3), the unit weight of water γ_w (kN/m^3), the difference in hydraulic head (water level) across the structure (ΔH) (m), and the coefficient of White (η=0.3).

The program's output comprises the following data: (1) the probability of failure due to piping, (2) the design point values of the parameters (will be discussed later), (3) the relative contribution of the parameters in the probability of failure, and (4) some other useful data related to the probabilistic analysis in the program. The calculated probability of failure will then be compared with the target probability of failure which is assessed at two percent of the overtopping risk. The structure will be safe against piping if the calculated probability of failure is less than the target probability of failure.

The design point value for each parameter, is a parameter point in the limit state function, with a minimum distance between the origin and the limit state in the multi-dimensional parameter space (Calle et al. 1989). Numerical methods to infer the design point parameter values have been developed and implemented in the FORM package used in the PIPING code.

3 PIPING ANALYSIS FOR THE SELECTED HYDRAULIC STRUCTURES IN URMIA REGION

3.1 *Collection of the prototype data*

Six diversion dams on Shahr-Chai, Nazlu-Chai, and Baranduz-Chai rivers in Urmia plain were the selected hydraulic structures to be investigated against piping. A typical cross section of the dams is shown in Figure 1. The geometrical dimensions of the diversion dams which included the seepage length in contact line of the structure with its foundation and the height of the structure were recorded using their design layouts and also by field measurements. Some soil samples from the foundation materials were collected and analyzed for their physical parameters. Table 1 shows the prototype data collected for the dams. The depth of the aquifer beneath the dams were obtained from the regional water authorities office in Urmia.

The bedding angle (θ) was 41 degrees and the drag factor ($\hat{C} = 4/\pi\eta$) was 0.3 for all soil materials. The hydraulic head across the structure (Table 1) which is the difference in potential head between the upstream and downstream of the dam, was calculated for each dam based on the maximum discharge in the river during a flood with the specified return period. For the Nazloo dam the design flood had a return period of 100 years and for the rest of the dams it was 50 years.

The standard deviations of all parameters were set at 10 percent of the mean values of the parameters (Table 1).

3.2 *Calculation of the probability of failure (p.o.f) for the dams*

Two cases were considered to calculate the probability of failure of the dams against piping. The first case uses the actual seepage length of the dams in their contact line with the foundation soil. The second case ignores the existance of the cutoff walls in the upstream side and the stilling basins in the downstream side of the dams. The second case simulates a situation in which due to poor construction quality or erosion effects, both the cutoff walls and the stilling basins break and be separated from the main overflow spillway part of the structure. This kind of breaks have been observed in some diversion dams constructed along the Urmia plain rivers.

Table 1. The prototype data collected for the diversion dams.

Name of the dam	ΔH^1 (m)	L^2 (m)	D^3 (m)	D_{70} (mm)	D_{10} (mm)	K (m/s)	γ'^4 (KN/m^3)
Nazloo	5.84	38.0	60	10.25	0.70	6×10^{-4}	9.8
Marangaloo	4.15	33.2	80	6.0	0.25	7×10^{-5}	11.7
Talatapeh	3.8	10.0	80	8.0	0.22	9×10^{-5}	11.7
Miavagh	6.0	5.0	20	10.4	0.46	8×10^{-4}	12.5
Kashtiban	5.2	19.0	50	7.0	0.30	6×10^{-4}	11.8
Nivloo	3.0	24.3	120	9.0	0.55	2×10^{-6}	9.9

1 Hydraulic head across the structure
2 Seepage length
3 Depth of the aquifer
4 Submerged unit weight of the foundation soil

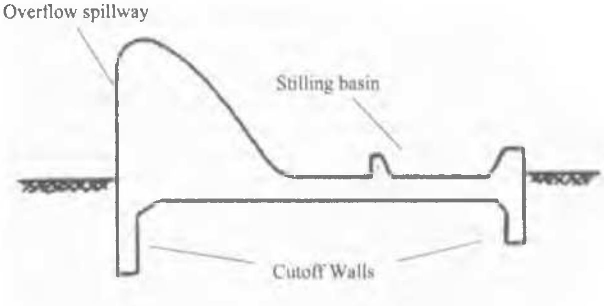

Figure 1 A typical cross section for the diversion dams.

Table 2. Calculated probability of failures of the dams due to piping for two cases considered.

Name of the dam	p.o.f (case 1)*	p.o.f (case 2)**	Target p.o.f
Nazloo	7.5×10^{-10}	1.4×10^{-1}	2×10^{-4}
Marangaloo	2.7×10^{-15}	1.3×10^{-2}	4×10^{-4}
Talatapeh	5.2×10^{-9}	6.4×10^{-3}	4×10^{-4}
Miavagh	0.445	0.75	4×10^{-4}
Kashtiban	5.2×10^{-5}	0.75	4×10^{-4}
Nivloo	6.2×10^{-23}	1.01×10^{-13}	4×10^{-4}

* Seepage length includes cutoff wall and stilling basin.
** Seepage length does not include cutoff wall and stilling basin.

The overtopping risk is the design return period of the structure for the maximum flood. Hence for the case of the Nazloo dam the target probability of failure due to piping was set to 2×10^{-4} and for the rest of the dams it was set to 4×10^{-4}. Table 2 shows the calculated probability of failures for the dams in two cases considered. As can be seen from the table, in the first case all dams are safe against piping except for the Miavagh dam. Due to a poor design, this dam does not have a cutoff wall and has a shorter stilling basin and hence, has a shorter seepage length. In the second case only the Nivloo dam is safe against piping and the other 5 dams have higher probability of failures than the target probability of failures. As indicated before, in this case it was assumed that due to the break of the cutoff wall and the stilling basin and separation of these elements from the overflow spillway, the seepage length has become shorter than the intact structure.

4 SENSITIVITY ANALYSIS OF THE PARAMETERS

Some sensitivity analysis was done to investigate the sensitivity of the PIPING model to different soil and geometrical parameters of the dams. Earlier sensitivity analysis performed by the first author using some prototype data for 4 sea dikes in the Netherlands, showed some minor irregularities in the model which were then fixed and the modified model produced reasonable results (Badv 1989). For further testing the accuracy of the PIPING model, some other sensitivity analysis was performed by the authors using the prototype data for 6 dams presented in this paper. Two series of analysis were performed. In the first series, mean value of a parameter (say seepage length L) was changed in the calculations by keeping the mean values and standard deviations of all other parameters constant. Then, the resulted probability of failures were plotted against the corresponding mean values of the parameter. As an example, Figure 2 shows the plot of the mean value of the seepage length against the probability of failure values for Nazloo diversion dam. It is evident from the figure that by increasing the seepage length of the dam, the probability of failure due to piping decreases.

In the second series of analysis, the standard deviation of a parameter was changed by keeping the mean values and standard deviations of all other parameters constant. Then the resulted probability of failures were plotted against the variation of the standard deviations. As an example, Figure 3 shows the results obtained for standard deviation of the seepage length for Nazloo dam. It is seen from the figure that by increasing the standard deviation of the seepage length, the probability of failure increases until it reaches to almost a constant value at higher standard deviations.

The sensitivity analysis produced valuable results from which

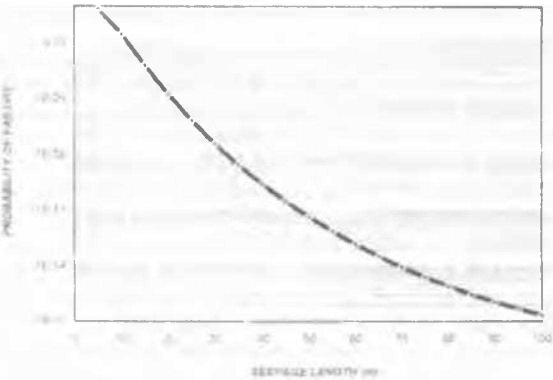

Figure 2. Variations of the probability of failure due to piping against the mean value of the seepage length in Nazloo dam.

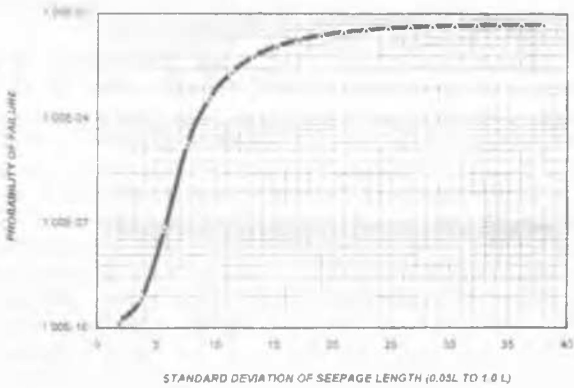

Figure 3. Variations of the probability of failure due to piping against the standard deviation of the seepage length in Nazloo dam.

the nature of each parameter (mean value or its standard deviation) in the overall safety of the structure against piping could be verified. These analysis did not show any irregularities in the results and further proved the accuracy of the PIPING model.

5 DETERMINATION OF THE PARTIAL SAFETY FACTORS

5.1 *Safety code*

The standard suggestion for safety control uses three basic factors of safety, namely γ_s applicable to loads and their effects, γ_m applicable to strength of the structure (elements), and γ_c which allows for mode type (sudden or nonsudden failure) and consequences of failure. The performance requirement to be met with regard to a limit state is:

$$S_{char} \cdot \gamma_s \cdot \gamma_c \leq R_{char} / \gamma_m \quad \text{or,}$$

$$R_{char} / \gamma_m - S_{char} \cdot \gamma_s \cdot \gamma_c \geq 0 \tag{7}$$

Where S denotes for solicitation (loading), R denotes for resistance (strength) and the suffix (*char*) refers to the characteristic quantities. In the case of the piping model, loading is associated with the occurrence of an unfavorable river head level and resistance is associated with the limit state head level. The characteristic river head level will be formulated in terms of a level with a specified probability of exeedence corresponding with the considered overall level of safety of the structure. So, the γ_c factor will be included in the characteristic load. The performance requirement for the piping analysis then reads:

$$\Delta H_{char} / \gamma_m - \Delta h_{char}\, \gamma_s \geq 0 \tag{8}$$

The partial safety factors of γ_m and γ_s involved in the reference designs for piping then read:

$$\gamma_m = \Delta H_{char} / \Delta H_{design} \tag{9}$$

$$\gamma_s = \Delta h_{design} / \Delta h_{char} \tag{10}$$

in which ΔH_{char} is the characteristic value of the limit state head level which is calculated by Equations 3 and 5 using the characteristic values of the parameters (Table 1). ΔH_{char} values for 5 diversion dams discussed in this paper, were calculated and are given in Table 3. Δh_{char} is the characteristic potential head difference corresponding to a flood with a specified return period which is discussed in the following section. $\Delta H_{design} = \Delta h_{design}$ is the design value of the head across the dam and can be calculated using the design formula (Equations 3 and 5) by adopting the design point values of the parameters produced by the PIPING program in the desired marginally safe configurations. This will be discussed in section 5.3.

5.2 *Estimation of the characteristic potential heads*

As indicated earlier, the target probability of failure due to piping is assessed at two percent of the overtopping risk for hydraulic structures. For the river head, the level corresponding with the specified overtopping risk is taken as the characteristic estimate. By subtracting the downstream water level from this value, the characteristic difference in water level (Δh_{char}) could be obtained. For the river discharges, a proper distribution is used (e.g., an exponential type of distribution). The relationship between the discharges and the probability of exceedence for each river in any country is determined by a committee and the maximum yearly discharge-water level relationship along the river could be obtained. For the Chahr-Chai, Nazloo-Chai and Baranduz-Chai river discharges, the Gumbel type of distribution is used by the regional water authorities. Using the 100 years return period for Nazloo dam and 50 years return period for the other dams, the Δh_{char} values were calculated for each dam and presented in Table 3.

5.3 *Calculation of design point values of the potential heads*

The PIPING program calculates the design point value of each parameter. To calculate $\Delta H_{design} = \Delta h_{design}$ for each dam, in the PIPING program, the seepage lengths (L, Table 1) with a constant range of standard deviations (i.e., 10% of the mean values) were adjusted until the desired marginally safe configurations were found (i.e., probability of failure of 2×10^{-4} for the return period of 100 years for Nazloo dam, and probability of failure of 4×10^{-4} for the return period of 50 years for the other dams). Then, the calculated design point values of the parameters were compared with the 90 percent confidence intervals of each parameter. For the parameters that their design point values lie within 90 percent confidence interval, the expected mean parameter values were taken as the characteristic values. For the parameters that their design point values lie outside this interval, either 5 or 95 percent exceedence level, depending on what is conservative regarding the piping analysis, was taken as the characteristic value. For the 5 dams under discussion, 5 percent confidence intervals of the parameters L, θ, η, and D_{70} were found to be the characteristic values. For the rest of the parameters the expected mean parameter values (Table 1) were taken as the characteristic values. Finally, by using these characteristic values in the Equations 3 and 5, the design value of the head was calculated for each dam (Table 3).

Table 3. Results of the partial safety factor calculations.

Name of the dam	$\Delta H_{design} = \Delta h_{design}$ (m)	ΔH_{char} (m)	Δh_{char} (m)	γ_m	γ_s
Nazloo	10.93	32.86	5.84	3.0	1.87
Marangaloo	7.71	39.93	4.15	5.18	1.86
Talatapeh	6.96	18.95	3.80	2.72	1.83
Miavagh	11.26	8.60	6.00	0.76	1.88
Kashtiban	9.90	17.05	5.20	1.72	1.90

5.4 *Calculation of the partial safety factors*

Using the data shown in Table 1, the partial safety factors (Equations 9 and 10), were calculated using the procedure mentioned above. The resulted partial safety factors ranged from 1.83 to 1.90 for γ_s, with the average of 1.87, and from 0.76 to 5.18 for γ_m, with the average of 2.68 (Table 3). Using these average values in Equation 8 will produce the following performance requirement for the piping analysis in Urmia region:

$$(\Delta H_{char} / 2.68) - (1.87\ \Delta h_{char}) \geq 0 \qquad (11)$$

The above requirement was obtained using the data from 5 diversion dams in Urmia region. More calculations is recommended to be done using more data from other dams to better estimate the partial safety factors for this region. Hence, for the time being, the above requirement should be used with caution. Similar calculations has been done by the first author using the data from 4 sea dikes in the Netherlands and the γ_s and γ_m values ranged from 0.9 to 1.04 and from 1.44 to 1.62, respectively and the average values of $\gamma_s = 1.1$ and $\gamma_m = 1.6$ were suggested for design purposes (Badv, 1989). For any specific region partial safety factors could be calculated using the prototype data obtained from that region. These safety factors then could be used for the Sellmeijer's formula to obtain safe characteristic difference in water level between the upstream and downstream of the structure to protect the structure against piping.

6 COMPARISON OF THE BLIGH, LANE, AND SELLMEIJER MODELS

The seepage lengths calculated with the Sellmeijer's method for the marginally safe configurations (see section 5.3), were compared with the seepage lengths calculated with the Bligh and Lane methods (Equations 1 and 2) using the data shown in Table 1. The following average ratios were obtained:

$$L_{Sellmeijer} / L_{Bligh} = 0.34 \qquad (12)$$

$$L_{Sellmeijer} / L_{Lane} = 0.49 \qquad (13)$$

The results indicate that the Bligh and Lane requirements for the seepage lengths may substantially be relaxed, allowing for smaller seepage lengths and economical designs. These ratios may differ in other locations.

7 SUMMARY AND CONCLUSIONS

Six diversion dams in Urmia region, Iran were controlled against piping for two different structural configurations. The PIPING model incorporating the Sellmeijer's analytical formula for piping along with the probabilistic approach was used in the analysis. The calculated probability of failures for piping compared with the target probability of failures and the dams which are not safe against piping were identified.

Sensitivity analysis was performed using the prototype data from the selected dams and further approved the accuracy of the model.

A procedure for determination of the partial safety factors for the Sellmeijer's formula, using the PIPING code and the prototype data, was presented and the partial safety factors were calculated for Urmia region. These calculations suggested the average value 1.87 for γ_s and 2.68 for γ_m which could be used in Urmia region for preliminary calculations. Further investigations for calculation of more reliable partial safety factors were suggested.

Comparison of the empirical methods of Bligh and Lane with the new analytical method of Sellmeijer showed that Sellmeijer method requires shorter seepage length for safe design and consequently produces economical design to protect dams against piping.

8 ACKNOWLEDGMENTS

Authors acknowledge the helpful suggestions and discussions made by Dr. J.B. Sellmeijer and senior engineer E.O.F. Calle at Delft Geotechnique, and senior engineer J.B.A. Weijers at Rikswaterstaat, in The Netherlands, throughout this study.

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