Numerical modelling of seepage analysis using SEEP/W: A case study for the Kerian River Flood Mitigation Project (Phase 3) in Bandar Baharu, Kedah

N F Mohd Nordin1,2*, H Mohamad1 and H Alarifi1

1 Department of Civil and Environmental Engineering, Universiti Teknologi PETRONAS, Bandar Seri Iskandar, 32610 Tronoh, Perak, Malaysia
2 Department of Irrigation and Drainage Malaysia, Jalan Sultan Salahuddin, 50626 Kuala Lumpur, Malaysia
*Corresponding author email: nor_16000645@utp.edu.my

Abstract. The levee stretch near Dataran Bandar Baharu was a part of the Kerian River Flood Mitigation Project (Phase 3) in Kedah to address frequent flooding problems at the area. It has been facing recurring levee failure despite several repair attempts in the past. As the research area was subjected to water level fluctuations, a numerical model using SEEP/W was analysed to study the effect of drawdown and seepage on the failed levee. Variables considered in this study include soil permeability, groundwater table and time dependent river water levels on the upstream side of the levee. The transient analysis carried out indicated that the levee was susceptible to failure due to the relatively high phreatic line within the levee during drawdown and a seepage face occurrence on the upstream side of the levee.

1. Introduction
With a total area of 1,621.4 km², the Kerian River basin is located predominantly in the state of Perak, with the northern portion of the Kerian River basin encroaching into the state of Kedah and mainland of Pulau Pinang. The 104 km Kerian River originates from the Bintang Range and flows south-westward towards the Malacca Straits near the Nibong Tebal town. Tributaries of the Kerian River include Selama River, Ijok River, Samagagah River and Semang River. After the 2014 major flood, a series of projects divided into 30 phases between the three states, namely Perak, Kedah and Pulau Pinang had been initiated by the Federal Government of Malaysia to prevent catastrophic flood reoccurrences in the Kerian River basin.

The Kerian River Flood Mitigation Project (Phase 3) located in Bandar Baharu, Kedah (figure 1) was one of the above-mentioned projects with its construction area starting from the Sungai Tepus vicinity until Ampang Jajar (Kerian River barrage). Despite project completion in early 2019, the 350 metres levee stretch between CH17,200 and CH17,550 faced persisting levee failure where the first indication of levee failure had occurred in August 2015. Since then, the levee failed separately in December 2015, February 2017, October 2017 and June 2018. Though several repair attempts had been carried out amounting to approximately RM400,000.00, the problem was not addressed justifiably. Slides and horizontal cracks along both the upstream and downstream slope of the levee was observed along the referred stretch. Figure 2 presents an aerial view of the failure and figure 3 depicts a close-up view of the failure.
Figure 1. The location of the Kerian River Flood Mitigation Project (Phase 3) in Bandar Baharu, Kedah.

Figure 2. Aerial view of the levee stretch between CH17,200 and CH17,550 for the Kerian River Flood Mitigation Project (Phase 3) in Bandar Baharu, Kedah.

Figure 3. Close up view of the failed levee stretch between CH17,200 and CH17,550 for the Kerian River Flood Mitigation Project (Phase 3) in Bandar Baharu, Kedah.
The Kerian River at this area was subjected to water level fluctuations due to the varying river water levels caused by changes in rainfall intensity, the river’s role in the agricultural sector for paddy fields irrigation; and its location nearby to the river estuary which causes tide-induced fluctuations. As the water level fluctuation at the area was moderate, it was possible that the initial design of the constructed levee was primarily based on steady state seepage analyses and precluded transient analyses. Hence, numerical modelling was carried out using SEEP/W to study the effect of drawdown and seepage on the failed levee.

2. Literature Review

Transient flow is defined as the condition during water flow where pore water pressure, and consequently total head, varies with time [1]. The change of the hydraulic boundary conditions and total stresses that occurs whilst in transient state causes: (i) saturated seepage through relatively pervious foundation strata [2], [3]; (ii) unsaturated seepage through earth embankments [4], [5]; and (iii) shear induced pore water pressures [6], [7]. The rate of pore water pressure response to these changes are mainly controlled by the hydraulic conductivity and compressibility of the soil [8].

Past literature regarding analyses on rapid drawdown include [6], [9]–[12] with many variations. Determination of the pore water pressure or the phreatic line through the levee is essential to further analyse its stability [13]. An increase of the pore water pressure will result in a decrease of the effective stress in the soil thus leading to a reduced factor of safety for the slope [14]. Most geotechnical engineering practice adapt a simplified approach to determine the phreatic line. However, complex numerical approach such as the finite element method provides a more thorough analysis [15], [16].

Numerical approach using SEEP/W have been utilised by past researchers to simulate both saturated and unsaturated flow in soils [17]–[19]. SEEP/W is a numerical model software that can mathematically simulate the real physical process of water flowing through a particulate medium using the finite element method; with results that are consistent with results from physical modelling in the laboratory [20]. Numerical modelling has multiple advantages over physical modelling whereby some of the evident advantages include a quick set up, readily available to investigate a wide range of scenarios, provides ease in accounting for gravity, ensure safety and prevents danger of physical harm to personnel as well as able to provide information and results at any location within the configured cross section [20].

3. Geometry and Material Properties

Survey works that was carried out previously before the commencement of the Kerian River Flood Mitigation Project (Phase 3) served as a basis for the model geometry. The survey cross section at CH17,400 in figure 4 was chosen as it was located approximately mid-length of the levee stretch experiencing failure, thus providing appropriate representative of the study area.

Adopting a scale of 1:250, polylines were drawn starting at the middle of the Kerian River riverbed up until the levee cross section on the right side of the riverbank. In accordance with the constructed levee on site, a 1V:4/3H slope was drawn for both upstream and downstream side of the levee, with a height of 1500 mm and crown width of 3000 mm. The subsurface foundation of the levee was divided into four layers based on laboratory test results of borehole samplings that had been carried out prior to this study. A sketch of the utilised SEEP/W model geometry is presented in figure 5 with the soil material properties for each layer defined in table 1.

The soils were classified using the British Soil Classification System where laboratory tests had identified the levee fill (Layer 1) as SILT with low plasticity; Layer 2, 3 and 4 as CLAY of extremely high plasticity; and Layer 5 as SAND with presence of fines. Laboratory tests had also identified the soil hydraulic conductivity and compressibility parameters for Layer 1, 2, 3 and 4 respectively. However, the soil sample retrieved for Layer 5 was inadequate to carry out advanced laboratory tests. Thus, correlated Standard Penetration Test (SPT) N-values and published literature values were used [21]–[24].
Figure 4. Survey cross section at CH17,400 which was located approximately mid-length of the levee stretch experiencing failure.

Figure 5. Model geometry of the study area utilised in SEEP/W.

Table 1. Soil material properties adopted for the SEEP/W model.

| Material Parameter | Layer 1 | Layer 2 | Layer 3 | Layer 4 | Layer 5 |
|--------------------|---------|---------|---------|---------|---------|
| Saturated conductivity (K), m/s | Levee fill material | BHF | BH G | BH G | Correlated, literature values |
| BS 5930 | FML | CE | CE | CE | S-F |
| Residual water content, m³/m³ | 3.00-3.50 | 6.00-6.50 | 9.00-9.50 | | |
| Saturated water content, m³/m³ | 1.42 x 10⁻⁸ | 7.77 x 10⁻¹⁰ | 7.91 x 10⁻¹⁰ | 7.37 x 10⁻¹⁰ | 1.68 x 10⁻⁴ |
| Coefficient of compressibility (Mv), kPa | 0.125 | 0.150 | 0.225 | 0.140 | 0.030 |
| | 0.2528 | 0.4290 | 0.4047 | 0.4272 | 0.4000 |
| | 0.0003000 | 0.0001833 | 0.0001767 | 0.0001767 | 0.0001000 |
4. Geometry and Material Properties

Seepage analysis was carried out using finite element in 2-dimensional with an element size of 0.5 m as its mesh configuration. An unstructured pattern mix of quads and triangles finite element mesh were created for the models. As this levee was modelled for transient analysis, time steps were created using river water levels that were recorded at the Kerian River barrage located downstream of the study area. Based on the chronology failure, the levee had shown its first signs of failure in early August 2015. Assuming that there were no drastic changes of the river water levels in the same months of the subsequent few years, river water level data from the 1st of August 2018 until the 7th of August 2018 were used. The maximum head of the river water level selected for modelling was limited to 2.51 metres above sea level (masl)* based on observation on site, taking into account the upstream location of the research area and the closure of the Kerian Barrage Gate during high water tide to prevent flooding. A boundary condition of zero pressure was also set at 500 mm below ground level nearby the downstream levee toe appertaining to visual site observation during boring and sampling.

Numerical modelling for the levee began with a steady state analysis at 2.28 masl followed by a transient drawdown to 0.87 masl to a transient increment to 2.51 masl, and so forth using the recorded river water levels in table 2. A total of 27 models were carried out where the river water level fluctuations were modelled for a duration of 5 to 7 hours each and seepage analysis were carried out at approximately 30 minutes interval with the river water level increment set linearly. Based on each recorded river water level, the level variation (Δ) was calculated and the phreatic line (represented by the water total head) development throughout the levee fill structure (toe-to-toe) were modelled (figure 6 and figure 7).

Table 2. Variation of the river water levels and phreatic line development.

| No. | Date       | Time     | River water level, H (masl) | Variation, Δ (m) |
|-----|------------|----------|-----------------------------|------------------|
| 1   | 1/8/2018   | 1.15 am  | 2.28                        | –                |
| 2   | 1/8/2018   | 7.14 am  | 0.87                        | -1.41            |
| 3   | 1/8/2018   | 1.07 pm  | 2.66*                       | +1.64            |
| 4   | 1/8/2018   | 7.55 pm  | 0.36                        | -2.30            |
| 5   | 2/8/2018   | 1.58 am  | 2.4                         | +2.04            |
| 6   | 2/8/2018   | 7.59 am  | 0.82                        | -1.58            |
| 7   | 2/8/2018   | 1.49 pm  | 2.69*                       | +1.69            |
| 8   | 2/8/2018   | 8.35 pm  | 0.30                        | -2.21            |
| 9   | 3/8/2018   | 2.40 am  | 2.47                        | +2.17            |
| 10  | 3/8/2018   | 8.44 am  | 0.81                        | -1.66            |
| 11  | 3/8/2018   | 2.30 pm  | 2.66*                       | +1.70            |
| 12  | 3/8/2018   | 9.15 pm  | 0.31                        | -2.20            |
| 13  | 4/8/2018   | 3.22 am  | 2.49                        | +2.18            |
| 14  | 4/8/2018   | 9.28 am  | 0.85                        | -1.64            |
| 15  | 4/8/2018   | 3.12 pm  | 2.56*                       | +1.66            |
| 16  | 4/8/2018   | 9.55 pm  | 0.38                        | -2.13            |
| 17  | 5/8/2018   | 4.05 am  | 2.45                        | +2.07            |
| 18  | 5/8/2018   | 10.14am  | 0.93                        | -1.52            |
| 19  | 5/8/2018   | 3.54 pm  | 2.41                        | +1.48            |
| 20  | 5/8/2018   | 10.35pm  | 0.52                        | -1.89            |
| 21  | 6/8/2018   | 4.50 am  | 2.37                        | +1.85            |
| 22  | 6/8/2018   | 11.03am  | 1.04                        | -1.33            |
| 23  | 6/8/2018   | 4.38 pm  | 2.21                        | +1.17            |
| 24  | 6/8/2018   | 11.18 pm | 0.70                        | -1.51            |
| 25  | 7/8/2018   | 5.39 am  | 2.26                        | +1.56            |
| 26  | 7/8/2018   | 12.00 pm | 1.17                        | -1.09            |
| 27  | 7/8/2018   | 5.28 pm  | 1.99                        | +0.82            |
Figure 6. Simulation of water total head during water level increment throughout the levee fill structure (toe to toe).

Figure 7. Simulation of water total head during drawdown throughout the levee fill structure (toe to toe).

No seepage face was observed at the downstream face of the levee for all of the models, indicating an acceptable outcome. However, it was noted that during drawdown, the lowering rate of the phreatic line was not concurrent with the decreasing river water level. This indicates presence of excess pore water pressures in soils above the river water level that affected the soil’s stresses and caused the levee failure. Figure 8 shows the plot of the pore water distribution throughout the levee fill structure (toe-to-toe) during water increment instances whilst figure 9 shows the plot of the pore water distribution throughout the levee fill structure (toe-to-toe) during drawdown instances. Positive pore pressure was observed for all the models during water level increment except for model 13 and 27 (after 3.09 days and 6.68 days respectively), which had shown a slight existence of negative pore water pressure beyond 300 mm from the levee toe. Contrarily for drawdown instances, negative pore pressure was observed in all the models within 400 mm from the levee toe with positive pore pressure observed beyond the 400 mm range. Negative pore pressure signifies the presence of partially saturated soils above the river water level, whereas positive pore pressure signifies fully saturated soils. This condition indicates a delayed pore water reduction as the water level decreased. A slow decline of the phreatic line
or the water table suggests a large lambda slope coefficient which can cause instability to the levee slope [25].

![Figure 8. Simulation of pore water pressure during water level increment throughout the levee fill structure (toe to toe).](image1)

![Figure 9. Simulation of pore water pressure during drawdown throughout the levee fill structure (toe to toe).](image2)

To better understand seepage and flow characteristics at the upstream side of the levee that experienced failure, a graph showing the rate at which water flux was accumulated on the upstream face area of the levee was plotted (figure 10). Water flux is defined as the volumetric flow rate per unit area per time. Based on Figure 10, the total water flux was influenced by the fluctuations of the river water level. When there is an increment of the river water level, the total water flux at the upstream face of the levee increased significantly. Subsequently, seepage flux also occurred during drawdown. However, the amount was substantially lesser, given that the river water level was near the base of the levee, reducing the unit area.

The results also showed a seepage face occurrence in all the drawdown models along the upstream side of the levee, represented by the velocity vectors as illustrated in figure 11. In SEEP/W, velocity vectors indicate where flow is occurring and its’ quantity. The presence of seepage at the face of the
levee upstream slope reduces its cohesive strength and alternately increases shear stress, causing failure on the levee structure. This is in conformance with the study carried out by [26] in developing an empirical relationship between the seepage forces acting on the side slope of a channel and the stability of the channel with various flow conditions.

![Figure 10. Total water flux at the upstream face of levee.](image)

![Figure 11. Illustrative example of the vectors generated in SEEP/W representing probable seepage face along the upstream side of the levee during drawdown.](image)

5. Conclusion
Seepage analysis was carried out using numerical models in SEEP/W for the failed levee stretch in the Kerian River Flood Mitigation Project (Phase 3) in Bandar Baharu, Kedah. A total of 27 models were developed using the finite element method to simulate the levee in transient conditions for an overall period of 7 days. Results indicated that during river water level increment, the seepage rate into the levee structure was noticeably higher compared to the rate at which it was seeping out during drawdown. The slow decline of the phreatic line within the levee structure during drawdown and the occurrence of seepage face at the upstream side of the levee reflects an excess in pore water pressure thus causing the levee susceptible to instability and failure.

6. References
[1] Lambe T W and Whitman R V 1969 Soil Mechanics (New York: John Wiley & Sons)
[2] Casagrande A 1961 Control of seepage through foundations and abutment of dam, first Rankine lecture Géotechnique 11 161–82
[3] Mansur C I, Postol G and Salley J R 2000 Performance of relief well systems along Mississippi river levees J. Geotech. Geoenvironmental Eng. 126 727–38
[4] Lam L, Fredlund D G and Barbour S L 1987 Transient seepage model for saturated–unsaturated soil systems: a geotechnical engineering approach Can. Geotech. J. 24 565–80
[5] Le T M H, Gallipoli D, Sanchez M and Wheeler S J 2011 Stochastic analysis of unsaturated
seepage through randomly heterogeneous earth embankments Int. J. Numer. Anal. Methods Geomech. 36 1056–76

[6] Duncan J M, Wright S G and Wong K S 1990 Slope Stability during Rapid Drawdown Proc. of the H. Bolton Seed: Memorial symposium pp 253–72

[7] Alonso E E and Pinyol N M 2011 Landslides in reservoirs and dam operation Dam maintenance and rehabilitation II pp 3–27

[8] Pauls G J, Sauer E K, Christiansen E A and Widger R A 1999 A transient analysis of slope stability following drawdown after flooding of a highly plastic clay Can. Geotech. J. 36 1151–71

[9] Bishop A W 1954 The use of pore-pressure coefficients in practice Géotechnique 4 148–52

[10] Moregenstern N 1963 Stability charts for earth slopes during rapid drawdown Géotechnique 13 121–31

[11] Lowe J and Karafiath L 1960 Effect of anisotropic consolidation on the undrained shear strength of compacted clays Research Conf. on Shear Strength of Cohesive Soils (ASCE) pp 837-58

[12] Baker R, Frydman S and Talesnick M 1993 Slope stability analysis for undrained loading conditions Int. J. Numer. Anal. Methods Geomech 17 15–43

[13] Yan Z L, Wang J J and Chai H J 2010 Influence of water level fluctuation on phreatic line in silty soil model slope Eng. Geol. 113 90–98

[14] Perri J F, Shewbridge S E, Cobos-Roa D A and Green R K 2012 Steady state seepage pore water pressures' influence in the slope stability analysis of levees GeoCongress 2012: State of the Art and Practice in Geotechnical Engineering pp 604–13

[15] Huang M and Jia C Q 2009 Strength reduction FEM in stability analysis of soil slopes subjected to transient unsaturated seepage Computers and Geotechnics 36 93–101

[16] Sun G, Yang Y, Cheng S and Zheng H 2017 Phreatic line calculation and stability analysis of slopes under the combined effect of reservoir water level fluctuations and rainfall Can. Geotech. J. 54 631–45

[17] Arshad I, Babar M M, and Vallejera C A E 2019 Computation of seepage and exit gradient through a non-homogeneous earth dam without cut-off walls by using Geo-slope (SEEP/W) software PSM Biological Research 4 40–50

[18] Aribudiman I N, Redana I W, Harmayani K D and Ciawi Y 2019 Seepage in soil from the difference of water viscosity using Geo-studio SEEP/W program Int. Research Journal of Engineering, IT & Scientific Research 5 15–26

[19] Zhang L L, Zhang L M and Tang W H 2005 Rainfall-induced slope failure considering variability of soil properties Géotechnique 55 183–8

[20] GEO-SLOPE International Ltd. 2012 Seepage modelling with SEEP/W (Alberta: GEO-SLOPE International Ltd.) pp 4-5

[21] Jorgensen D G 1980 Relationships between basic soils-engineering equations and basic ground-water flow equations (US Department of the Interior, Geological Survey)

[22] Ng C W W and Shi Q 1998 A numerical investigation of the stability of unsaturated soil slopes subjected to transient seepage Computers and Geotechnics 22 1–28

[23] Wilson G V, Periketi R K, Fox G A, Dabney S M, Shields F D and Cullum R F 2007 Soil properties controlling seepage erosion contributions to streambank failure Earth Surface Processes and Landforms: The Journal of the British Geomorphological Research Group 32 447–59

[24] Chu-Agor M L, Wilson G V and Fox G A 2008 Numerical modeling of bank instability by seepage erosion undercutting of layered streambanks J. Hydrol. Eng. 13 1133–45

[25] Zheng Y R, Shi W M, Kong W X and Lei W J 2005 Determination of the phreatic-line under reservoir drawdown condition Slopes and Retaining Structures Under Seismic and Static Conditions pp 1-10

[26] Burgi P H and Karaki S 1971 Seepage effect on channel bank stability J. Irrig. Drain. Eng. 97 59–72
