Performance Monitoring of Restraint Back-to-Back Mechanically Stabilized Earth Walls for Dam Crest Rehabilitation of Mae Suai Dam, Thailand

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ABSTRACT: The Mechanically Stabilized Earth walls (MSE walls) for dam crest rehabilitation was constructed at Mae Suai dam, Thailand. Welding mesh gabion was used as the facing on both sides, the polymetric geogrid and rebar were used as reinforcements. Furthermore, the MSE walls was placed on the original earth dam crest, steel sheet pile was installed at upstream and downstream side to prevent leakage and control the settlement of the new dam crest. The instruments were installed at various test sections to careful field monitoring to obtain high-quality data. The results obtained from 2D finite element method simulations were in good agreement with the field measurements, the lateral deformation and settlements were very small. The axial forces in rebar reinforcement were found to be continually changing due to deformation of foundation, external stimuli and construction factors. Likewise, the strain measured in all positions of geogrid reinforcement was very low. Combining steel reinforcement (high stiffness) with geogrid reinforcement (low stiffness) was redundant. Most of the lateral stresses are resisted by the former than the later. It can be concluded that in a reinforced soil wall that uses two or more types of reinforcing materials, tensile force is developed in higher stiffness material.

KEYWORDS: Dam crest rehabilitation, Dam crest raising, MSE Walls, and Reinforced earth embankment.

1. INTRODUCTION

The reinforced earth embankment namely, Mechanically Stabilized Earth walls (MSE walls) or Geosynthetic reinforce soil walls (GRS walls) are used extensively as earth retaining structures because of their cost-effectiveness and ability to withstand much larger differential settlements than conventional reinforced concrete retaining walls (Kim *et al.*, 2012; Watanbe *et al.*, 2003). Various types of MSE wall facings and reinforcements are used depending on the specific application, soil conditions and wall. Ho and Rowe (1996) found that the reinforcement stiffness, vertical spacing and length to wall height ratio, L/H, are important parameters that influence the wall displacement response. Rowe and Ho (1998) showed that the magnitude of wall lateral displacement is influenced by the soil friction angle and a reinforcement stiffness factor. The simplified design and analysis methods of reinforced earth embankment are provided in design guidance documents such as BS8006 (BSI, 2010) in the UK, AASHTO (AASHTO, 2012) and FHWA (Berg *et al.*, 2009) in the USA. Issues related to the design and factors affecting the performance of reinforced soil have been addressed by many researches in recent times (Allen *et al.*, 2004; Allen *et al.*, 2003; Bathurst *et al.*, 2008; Bathurst *et al.*, 2009; Bathurst *et al.*, 2006; Huang *et al.*, 2010; Kongkitkul *et al.*, 2007; Leshchinsky, 2009; Miyata *et al.*, 2015). Also, the behavior of reinforced earth structures has been comprehensively studied through field observation of full-scale physical model, laboratory model testing, and numerical simulation. Shivashankar (1991) observed the behavior of a welded wire wall with poor quality, cohesive-friction backfills on soft Bangkok clay. Voottipruex (2000) studied the behavior of full-scale embankment built in AIT campus which was reinforced with hexagonal wire mesh up to 6 m with 10° inclined of gabion facing. Bergado *et al.* (2000) simulating the behavior of the full-scale test embankment were the method of applying the embankment loading during the construction process, the variation of soil permeability during the consolidation process, and the selection of the appropriate model and properties at the interface between the soil and reinforcement. Holtz and Lee (2002) made report on research conducted on the internal stability of reinforced soil walls. Using the results of monitoring of 6 walls with different reinforcement elements and types of backfill material, they made recommendations for improving the modeling techniques for the level of working stress. Bergado *et al.* (2003) analyzed the behavior of reinforced embankment with silty sand backfill built on soft soil. The embankment was reinforced with galvanized and PVC coated hexagonally shaped geogrids. Bergado and Teerawattanasuk (2008) compared the effect of embankment geometry with 2D and 3D simulations and concluded that 3D analysis must be conducted for short embankments to obtain good agreement with measured field data. Huang *et al.* (2009) has investigated different soil constitutive models and their influences on the results. The paper confirmed that the modified Duncan–Chang model is a suitable constitutive model and that the parameters used in that model can be determined from conventional triaxial testing. Baral *et al.* (2016) compared the behavior of polymeric and metallic reinforced embankments on hard foundation with 3D numerical simulations conducted using PLAXIS 3D. The lateral displacements and settlements were very small in the case of the MSEW with inextensible reinforcement. The corresponding lateral and vertical deformations in the RSS were much larger due to its extensible reinforcing materials. Furthermore, many researchers have studied the behavior of Back-to-Back MSE walls (Benmebarek *et al.*, 2016; Benmebarek and Djabri, 2017; El-Sherbiny *et al.*, 2013; Lajevardi *et al.*, 2021; Samee *et al.*, 2021; Xu *et al.*, 2021; Yang *et al.*, 2022). However, a few research has studied the reinforcing retaining walls used for dam crest rehabilitation/ raising. Hardianto Fransiscus *et al.* (2013) describes the design method and construction challenges of a geosynthetic-strip-reinforced mechanically stabilized earth (MSE) wall used for the expansion of Los Vaqueros reservoir dam in Contra Costa County, CA. The site is located in a high seismic area, and with their proven performance under such conditions, an MSE wall with a maximum height of 15m was designed and installed to provide a wider dam crest while being part of the embankment system to increase the dam height by 10.4m.

A 6-m-high reinforced earth embankment for dam crest rehabilitation was constructed at Mae Suai dam, Thailand. Welding mesh gabion was used as the facing on both sides, the polymetric geogrid and rebar were used as reinforcements, steel sheet pile was installed at upstream and downstream side. This embankment was fully instrumented with piezometers, settlement plates, inclinometers and strain gauges and subjected to careful field monitoring to obtain high-quality data. In this research, the field measurement data was verified with 2D FEM analysis using MIDAS GTS to determine the performance of back-to-back MSEW for dam crest rehabilitation. Particular attention was given to the lateral displacements, vertical settlements and axial forces in the reinforcements.

2. MAE SUAI DAM AND DAM REHABILITATION

Mae Suai dam is a 59 m high structure with 400 m crest length and having a reservoir capacity of 73 million cubic meters (Figure 1). The dam has been in operation since 2003. The RCC section is used as an overflow spillway and was designed for 500 years return period of flood. The RCC material is a low paste RCC covered with the conventional concrete (CVC). Figure 2 shows the longitudinal and transverse section of the dam. The RCC section consists of an overflow spillway and gravity retaining wall at both sides to create flow channels and retain the earth dam at both sides.

Figure 1 Mae Suai Dam (Soralump *et al.*, 2023)

The RCC section is surrounded by earth zone dam. Core trench of the earth zone dam was excavated to the foundation rock in the river bed and abutments. Impervious clay consisted of low-plasticity clay (CL) and internal filter (sand and gravel) consisted of clayey sand (SC) materials to reduce the water pressure and discharge the seepage water into RCC gallery. Shell zone or random zone is made up of semi-impervious coarse grain earth (low-plasticity clay (CL), clayey sand (SC) and silty sand (SM)) with horizontal drain to drain out the water during drawdown period and maintain the stability of shell zone. The earth zone extends in both side of the abutment. The 6 m high RCC retaining block was constructed over the earth filled material at the downstream of dam crest to reduce the earth fill work on downstream slope and lower the construction cost.

The transition trapezoidal RCC block (Block D) was constructed near the joint between RCC spillway section and earth zone dam. Furthermore, to prevent the erosion at the crest of earth dam during the overtopping of spillway, RCC blocks A, B and C were constructed as a water guide wall (wing wall). These blocks were placed directly over the earth filled material (Soralump *et al.*, 2023).

In 2004, after 1 year of operation of Mae Suai Dam, water overflowed the spillway and leakage was observed at the downstream crest in contact area between earth fill dam and RCC spillway structure. The water flow was clearly observed behind the RCC block where differential settlement was also clearly visible. The leakage was observed when reservoir reached a certain elevation near the dam crest. The repair work has been done by installing the impervious membrane over the surface of RCC blocks. The leakage flow was reduced after the repair of dam but did not disappear completely (Soralump *et al.*, 2016).

Royal irrigation department (Thailand) decided to rehabilitate Mae Suai dam and repair works was proposed to solve the leakage problem and stability of the dam during future earthquake. It has been proposed to remove the RCC blocks A, B, C, D and blocks on the crest of the earth dams (Figure 3) and replace RCC block on the downstream crest by more flexible structure. In this case, MSE Walls (MSEW) will be used (Figure 4), so that there won't be any rigid and brittle that crack when subjected to seismic force or differential settlement. Likewise, no further significant displacement, both vertically and horizontally, will be observed from the load of a new dam crest and it will be able to control the normal seepage and prevent leakage along the joints.

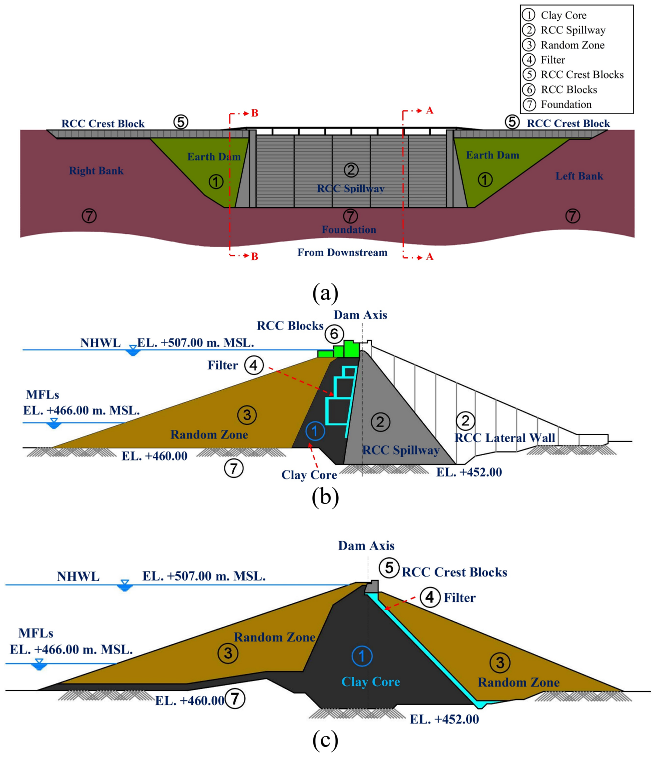


Figure 2 Longitudinal and Cross Section of Mae Suai Dam: (a) Longitudinal section; (b) Cross section A-A; (c) Cross section B-B (Soralump *et al.*, 2023)

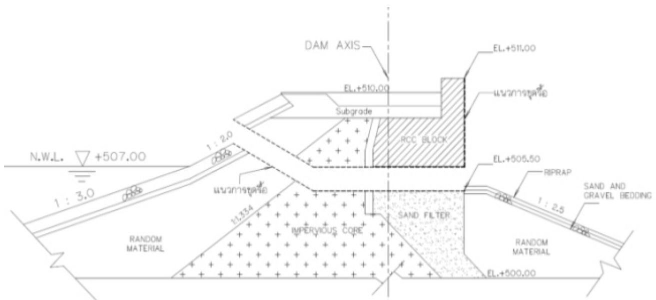


Figure 3 Removing of the RCC block and dam crest (Soralump *et al.*, 2016)

3. DESCRIPTION OF THE EMBANKMENT

A 6-m-high reinforced earth embankment for dam crest rehabilitation was designed by Geotechnical Engineering Research and development center (GERD) and constructed by Kanber Geotechnic (Thailand) Company limited with construction supervision by the Royal Irrigation Department (Thailand) and Samart Engineering Consultants Company limited acted as a project consultant. The MSE wall was 177 m along the dam crest and 10.90 m wide at the top and was prepared by welding the mesh Gabion 1.20x1.20x1.20 m as the facing on both sides and fastened with rebar (DB16 mm) between the gabion. The steel plate of dimension 200x200x9 mm and nut were used for the connection joints as shown in Figure 4 and Figure 5. In addition, the polymetric geogrid reinforcement was added to the MSE wall. The spacing of the rebar reinforcement was 0.60 m. in vertical and horizontal directions. Likewise, the vertical spacing of the geogrid reinforcement was 0.30 m. Furthermore, the new MSE walls will be placed on the original dam crest that consist of 3 materials type: 1) Impervious Core 2) Filter and 3) Random Zone. Steel sheet piles of 10 m and 6 m length were installed at the upstream and downstream sides to prevent leakage and control the settlement of the original earth dam crest.

3.1 Preliminary Design of MSE Walls/Embankment

Preliminary design of the Back-to-Back MSE wall was carried out based on LRFD Method (AASHTO, 2012; Berg *et al.*, 2009). The external and internal stability has been analyzed according to the geometry of the wall. Sand and gravel available around the construction site is used as backfill material. The wall height assumed for the preliminary design is 6 m and the incline facing was 18.30 degrees from the vertical.

The internal stability, tension in the reinforcement behind the failure surface was checked against the lateral internal earth pressures on the assumption that each side of the wall was independent. In the design, the designer has divided the behaviour of reinforcement load into two parts: 1) During construction and static loads, the geogrid is defined as a reinforcing material, failure surface is determined by both the coherent gravity method and the coulomb method concurrently. 2) seismic loads, rebar is defined as a reinforcing material, failure surface is determined by the coherent gravity method. Furthermore, finite element analysis was performed to determine the tensile force in both reinforcing materials. The external stability was examined using the 2D finite element analysis by strength reduction method (SRM) as mentions in the next section

3.2 Construction Method

After the removal of original dam crest (Figure 3), steel sheet piles of depth 10m and 6m were installed at the upstream and downstream sides. Site clearing and levelling works were carried out for the marking of the position of the proposed MSE wall/embankment.

At first, drainage system was installed for the new dam crest and Geosynthetic Clay Liner (GCL) was laid over the wall foundation to prevent the seepage of water into the earth dam from above. The welding mesh was placed along the dam crest on two sides and filled the gabion with 30 cm high rock. The first layer of geogrid across the embankment from the upstream gabion to the downstream gabion was installed (Figure 10). The geogrid was installed and backfill layer was compacted layer by layer until the height of wall was reached (Figure 7). At the back of the gabion, geogrid was folded and geotextile was installed to prevent the backfill leaked out into the gabion (Figure 5). Longitudinal and transverse rebars were installed through the gabion and attached to the gabion by steel plate and nut (Figure 5 and Figure 6). While installing the gabion up to the 3rd layer, the clay between MSE wall and the sheet pile was compacted as shown in Figure 4. Finally, the upper surface with asphalt having thickness of 0.10m shall be used as a traffic surface. During the construction of the embankment, field density test at various selected places were carried out using sand cone replacement method to ensure compaction was carried out to minimum of 95% standard proctor density.

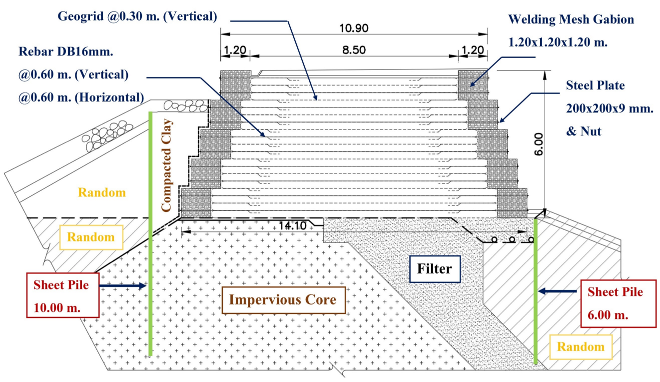


Figure 4 Restraint Back-to-Back MSE Wall (Soralump *et al.*, 2016)

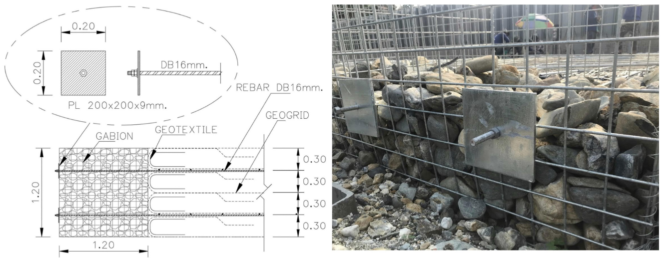


Figure 5 Connection joints between the rebar and gabion

Figure 6 Rebar reinforcements installation

Figure 7 Construction of full scale MSE wall

4. DAM INSTRUMENT

The rehabilitation of the dam crest using the MSE wall using the additional instruments was carried out to investigate the behavior of the new dam crest. Figure 8 shows the locations of test section with instrumentation selected from the location with the most settlement at the left and the right bank (Soralump *et al.*, 2023) and the position predicted that the wall would be least affected by boundary conditions (Plane strain). The instrumentation consists of Inclinometer (INC), Settlement Plate (SP), Piezometer (PI), Rebar Strain Gauges (RSG), Geogrid Strain Gauges (GSG) and Surface Monument (SM) as shown in Figure 9. Figure 10 shows the locations of test section with field instrumentation at the left bank. Inclinometers were installed on the side of the MSEW outside the steel sheet pile (at upstream and downstream section) to determine the lateral deformation of the new dam crest. Settlement plates were installed inside the MSEW at impervious core and filter to determine the vertical settlement of the wall foundation and settlement plates were installed inside the walls facing on both sides. Vibrating Wire (VW) Piezometer were installed at the foundation of the new dam crest between sheet pile and MSE wall to monitor the water pressure behind the sheet pile. In the past, VW Piezometer was installed near the contact between RCC Section and earth dam section that has leakage (Soralump *et al.*, 2023). Vibrating Wire Rebar strain gauges were installed along the length of the rebar at the designed location based on the failure surface constructed using the coherent gravity method to determine the axial forces generated in the rebar and internal stability of MSEW as shown in Figure 11a. Ultrahigh-elongation Foil Strain gauges (Figure 11b) were installed along the length of the geogrid reinforcement at the designed location based on the failure surface constructed using coherent gravity method and coulomb method to determine the tensile strain generated in the geogrid. The Surface Monument was also installed on the traffic surface above the new dam crest and on the bridge over the RCC section to monitor the settlement along the dam crest. V-Notch Weir was installed at the downstream side to monitored the drainage of the MSEW and leakage.

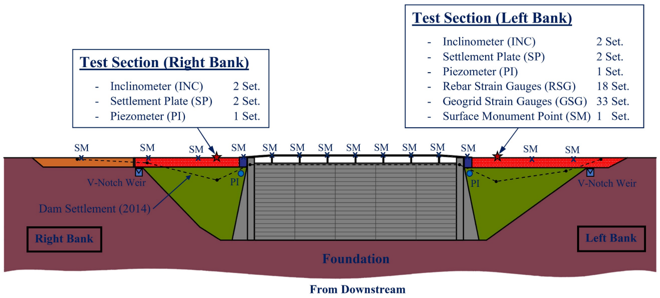


Figure 8 Location of test section with instrumentation (left and right bank)

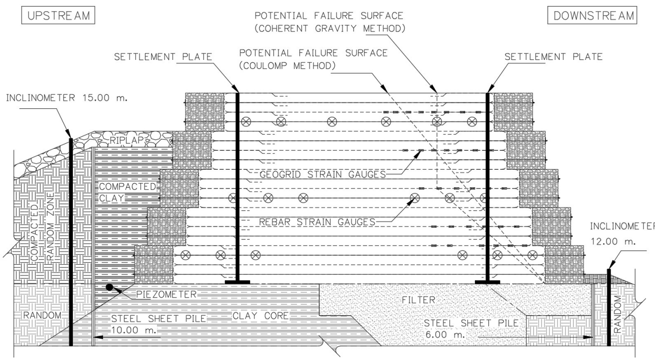


Figure 9 Schematic diagram of the test section with instrumentation (Left Bank)

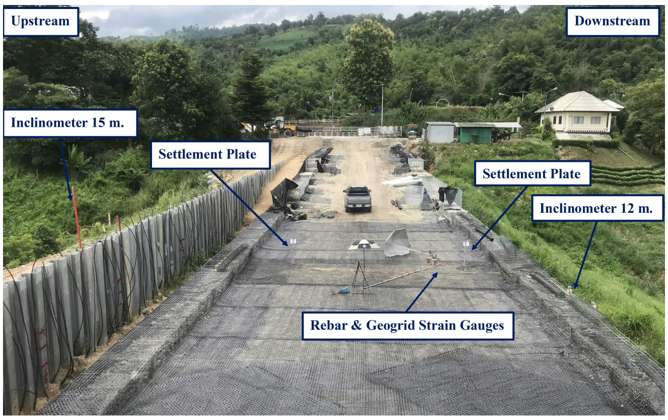


Figure 10 Geogrid installation and instrumentation of test section (left Bank)

Figure 11 Rebar and Geogrid strain gauges installation (a) rebar strain gauges (b) geogrid strain gauges

5. 2D FINITE ELEMENT MODEL OF MSE WALLS FOR DAM CREST REHABILITATION

The finite element analysis was performed using a geotechnical analysis software MIDAS GTS. In this research, 2D finite element model of original earth dam and MSE Walls for dam crest rehabilitation of Mae Suai Dam was created according to the dam geometry. Stress-Seepage-Slope coupled analysis was used in the analysis as there was sequential seepage-stress analysis and slope stability analysis during the construction process (Midasgts, 2018). Likewise, the slope stability analysis by strength reduction method (SRM) was used to determine the safety factors of MSEW for dam crest rehabilitation. Figure 12 shows the solution algorithm for 2D Finite element analysis of MSEW for dam crest rehabilitation. At first, the stress and deformation of the original earth dam was analyzed using Stress-Seepage coupled analysis (Figure 13 and Figure 14) followed by transient seepage analysis. The function of water level change was determined from the water retention record in first 3 years. The water level on upstream side was varied from the Minimum Water Level (MFLs) to the Normal High Water Level (NHWL) (Figure 2). During the first filling of a reservoir, the water level was increased from MFLs to NHWL within 1 year and the water level fluctuate during the period of storage in other years of operation. Likewise, total of 14 construction stage (1 year/stage) with time step of one month was used in analysis (Soralump *et al.*, 2023). In the next construction stage, the original dam crest elements in the parts has been removed (Figure 3) and clear displacement was set to zero preserving the stress history and deformation shape and it was replaced with new dam crest rehabilitated elements (MSE walls, new embankment and sheet piles) as shows in Figure 15 by setting the water level to MFLs. Finally, the safety factor of new dam crest was determined by Strength Reduction Method. Mohr-Coulomb model was used for the foundation, filters and RCC materials while Modified Cam-clay model was used for random and core of the dam. Furthermore, linear elastic model was used for reinforcement and steel sheet pile.

5.1 Geometry Model

2D finite element modelling was performed under plane-strain conditions. The geometry and height of the original earth dam was selected from the cross-section of the original earth dam at the test section (Figure 8) which was consistent with the FEA results by Soralump *et al.* (2023) as shown in Figure 13 and Figure 14. To avoid the boundary effects caused by the constraints of the numerical model, the foundation of the upstream and downstream embankment has been extended 120 m in both directions, twice the maximum height of the dam (Gikas and Sakellariou, 2008; Soralump *et al.*, 2023). Therefore, the deformation of the dam body had very little impact due to the constraints of the model. The plain-strain triangle and quadrilateral elements type were used in RCC and soil elements, the beam and embedded beam elements type were used in reinforcement (rebar & geogrid) and steel sheet piles. Element sizes varies from 0.10m to 20m, the smaller element size is located at the original dam crest and MSE Walls and increases when its lowered. The contact surface between steel plate and gabion are defined as plate bearing joints because each part can slide and separate.

Therefore, to transfer the compression, rigid link type interface elements have been used. The rigid link was a high stiffness element that is only able to transfer compressive forces in a horizontal direction and the movement of side surface was allowed in vertical direction Figure 15 shows the 2D Finite Element Model of MSEW for dam crest rehabilitation.

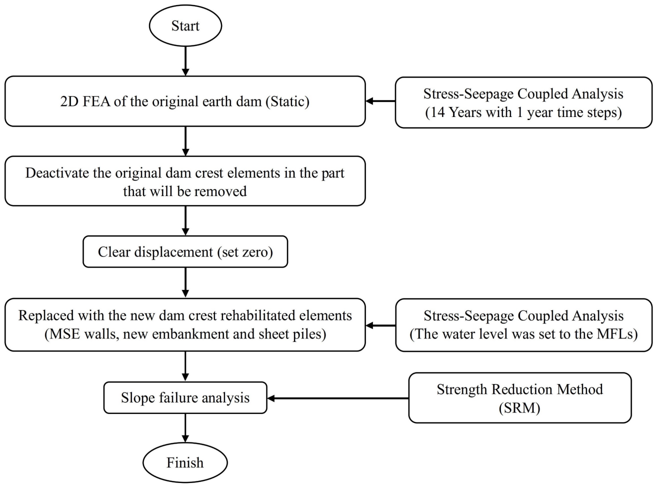


Figure 12 Solution algorithm for 2D Finite element analysis

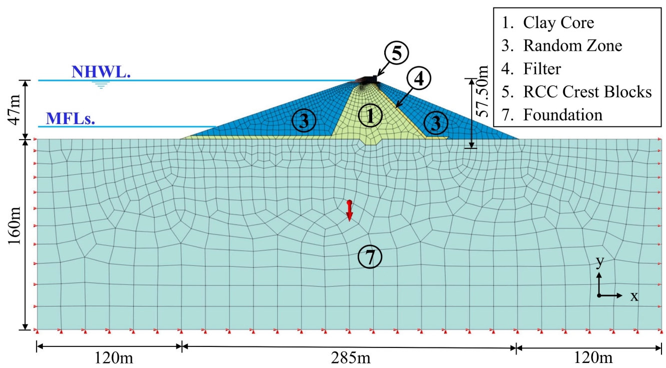


Figure 13 2D Finite Element Model of original earth dam, 57.50 m. high

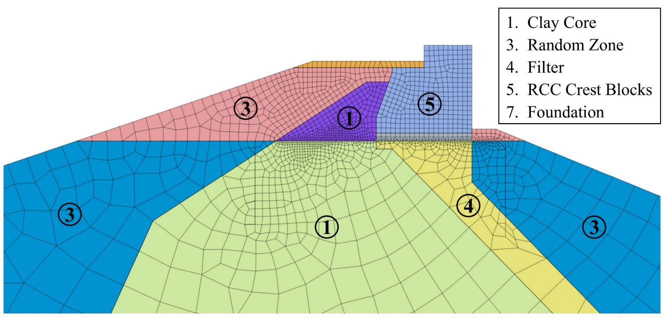


Figure 14 2D Finite Element Model of original dam crest

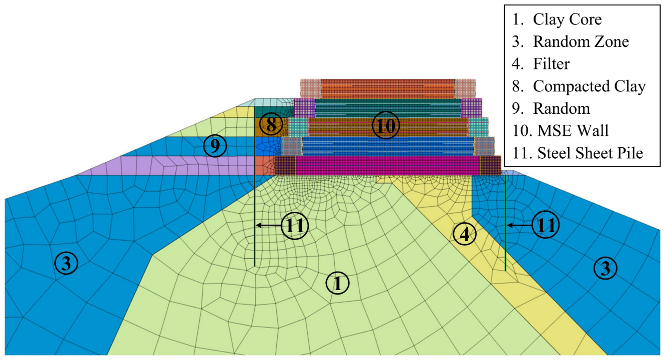


Figure 15 2D Finite Element Model of MSE Wall for dam crest rehabilitation

5.2 Geotechnical Parameters of the Materials

The foundation of MSEW for dam crest rehabilitation was the original earth dam that that has been in used for a while. Therefore, the foundation of the new dam crest consists of the components of the original earth dam, namely Impervious Core, Random Material, Filter, Dam Foundation and RCC. Some rudimentary information concerning the material parameters of foundation of MSE Walls used in the analyses were taken from Final design report of Mae Suai Dam (RID, 1998). The parameters were determined from the results of the geotechnical tests done on the samples from the borrow areas which has been identified during the geological survey of the site (Soralump *et al.*, 2023). The parameters of the original dam crest required for Midas GTS analyses are tabulated in Table 1. The backfill materials used in this embankment consisted of sand and gravel available near construction site and was classified as well graded gravel (GW). The welding mesh gabion wall-facing system was made from hot-dipped galvanized rectangular wire mesh (RB 6 mm.), uniform square mesh 8x8 cm. with tensile strength (f_y) of 2,830 kg/cm^2 (Figure 5), baskets size 1.20x1.20x1.20 m and filled with river rock of size 50 – 300 mm. The properties of the backfill material required for Midas GTS analyses are tabulated in Table 2. Two types of reinforcement, namely Geogrid and Rebar reinforcement were used in the reinforced embankment. The geogrid reinforcement was a high-strength steel wire composite with polyethylene with the aperture dimension of 5 x 5 cm (Figure 16). The material property was obtained from the wide-width strip tensile strength test for geogrid along the machine and along the cross-machine directions. The tensile strength along the machine and along the cross-machine directions were 60.41 KN/m and 56.56 KN/m respectively. The Elongation at break were 3.49% and 4.44% respectively. The rebar reinforcement was a deformed steel bar (DB16mm.) with tensile strength (f_y) of 4,440 kg/cm^2. The properties of the reinforcement are tabulated in Table 3. The rebar and geogrid reinforcement were considered to be a linear elastic material. Steel sheet pile was used in that dam crest rehabilitation (Figure 17). It was made from the hot-rolled steel and hot-dipped galvanized and was installed using vibration machine at the upstream and downstream side along the dam crest. The properties of the steel sheet pile are tabulated in Table 3. The steel sheet pile was considered to be a linear elastic material. The interface coefficients were used in the interaction between the backfill soil and the reinforcing materials, namely rebar, geogrid and steel sheet pile. The model parameters at the soil–structure interface can be generated from the soil using the strength reduction factors (R_{inter}), defined as the ratio of the shear strength of the soil–structure interface to the corresponding shear strength of the soil. In this research, the strength reduction factors (R_{inter})suggested by Brinkgreeve and Shen (2011) were used as shown in Table 4.

Table 1 Material properties of original dam used in FEM analyses

Name	Clay Core	Random Zone	Filter	Foundation	RCC
Model Type	Modified Cam Clay	Modified Cam Clay	Mohr Coulomb	Mohr Coulomb	Mohr Coulomb
Elastic Modulus, E (ton/m^2)	3,000.00	3,000.00	5,000.00	300,000.00	1,000,000.00
Poisson's Ratio (ν)	0.30	0.30	0.30	0.20	0.20
Unit Weight, γ (ton/m^3)	2.02	2.02	1.85	1.94	2.40
K_x (m/sec)	2.0×10^{-7}	2.0×10^{-6}	1.0×10^{-4}	3.0×10^{-7}	1.0×10^{-7}
K_y (m/sec)	5.0×10^{-8}	1.0×10^{-6}	1.0×10^{-4}	3.0×10^{-7}	1.0×10^{-7}
Cohesion (ton/m^2)	-	-	-	50.00	30.00
Frictional Angle, ϕ (Deg.)	-	-	35	40	40
Over Consolidation Ratio, OCR	1.000	1	-	-	-
Slope of NCL, λ	0.335	0.761	-	-	-
Slope of URL, κ	0.053	0.096	-	-	-
Slope of critical state line, M	0.856	1.02	-	-	-

Table note: NCL = normal consolidation line, URL = Unload-reloading line (Overconsolidation line)

Table 2 Properties of backfill material used in FEM analyses

Name	Backfill	Rock Fill
Model Type	Mohr Coulomb	Mohr Coulomb
Elastic Modulus, E (ton/m^2)	5,000	5,000
Poisson's Ratio (ν)	0.30	0.30
Unit Weight, γ (ton/m^3)	2.07	1.95
Kx (m/sec)	1.0×10^{-4}	1.0×10^{-4}
Ky (m/sec)	1.0×10^{-4}	1.0×10^{-4}
Cohesion (ton/m^2)	-	-
Frictional Angle, ϕ (Deg.)	41	41

Table 3 Properties of reinforcing materials used in FEM analyses

Name	Rebar	Geogrid	Sheet Pile	Welding Mesh	Steel Plate
Model Type	Linear Elastic	Linear Elastic	Linear Elastic	Linear Elastic	Linear Elastic
Elastic Modulus, E (ton/m^2)	2.04×10^7	28,550	2.04×10^7	2.04×10^7	2.04×10^7
Poisson's Ratio (ν)	0.30	0.45	0.30	0.30	0.30
Unit Weight, γ (ton/m^3)	7.85	0.93	7.85	7.85	7.85
Thickness (mm)	-	1	126.33	-	9
Diameter (mm)	16	-	-	6	-

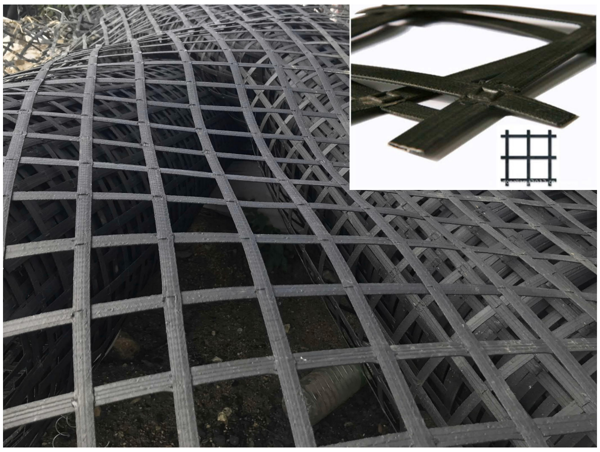


Figure 16 The geogrid reinforcement

Figure 17 The hot-dipped galvanized steel sheet pile

Table 4 Strength Reduction Factors (R_{inter})

Interaction	R_{inter}
Backfill / Rebar	0.60
Clay / Steel sheet pile	0.50
Backfill / Geogrid	1.00

6. RESULTS AND DISCUSSIONS

6.1 Vertical Displacement

Settlement plates were installed in the embankment (Figure 8, Figure 9 and Figure 10) to measure the vertical settlements and the results were verified with 2D FEA. Figure 18 shows the vertical displacement contour of original earth dam obtained from Stress-Seepage coupled analysis at the end of time step (2016) with maximum vertical displacement of 0.70 m. The results from the analysis are consistent with the geodetic monitoring data and FEA results by (Soralump *et al.*, 2023). Figure 19 and Figure 20 shows the vertical displacement contour of dam crest rehabilitation with MSE Walls and steel sheet pile after deactivating the original dam crest elements and replacing with the new dam crest elements. In other words, it was the settlement caused by the new dam crest. The maximum vertical settlement was found to be 0.014 m at the top surface of the backfill on the upstream side. Settlement of the foundation of MSEW according to the location of the settlement plate installed was 8 mm and 4 mm at the upstream and downstream sides, respectively. The settlement pattern slightly tilted to the upstream side. However, settlement of the foundation obtained from the FEA was small compared to the height of the dam (59 m.) or even the height of the MSEW (6m.) because the foundation of the MSEW supported the weight of the old dam crest which was heavier than the new dam crest. The analytical results are consistent with the measured results from the settlement plate as shows in Figure 21. The settlement occurred only at beginning of the wall construction and at the end when the wall height was 0.90 m. for SP1& SP3 and 1.50 m. for SP4. Likewise, no settlement was detected in SP2. The maximum settlement measured at position SP4 was 0.05 m. It can be seen that the settlement that occurred was an immediate settlement caused by construction conditions (insufficient preparation of the foundation surface), especially in the area where the measuring equipment was installed.

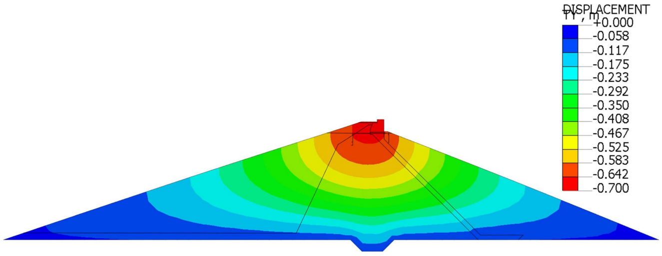


Figure 18 Vertical displacement of original earth dam

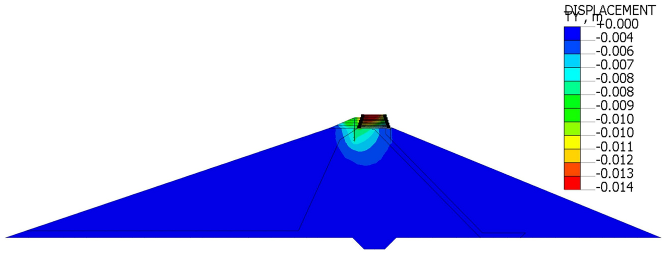


Figure 19 Vertical displacement of earth dam rehabilitated with MSE walls

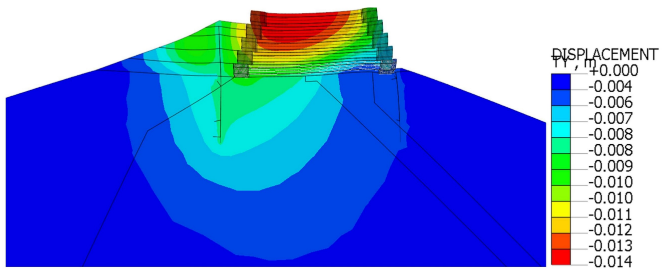


Figure 20 Vertical displacement of earth dam rehabilitated with MSE walls

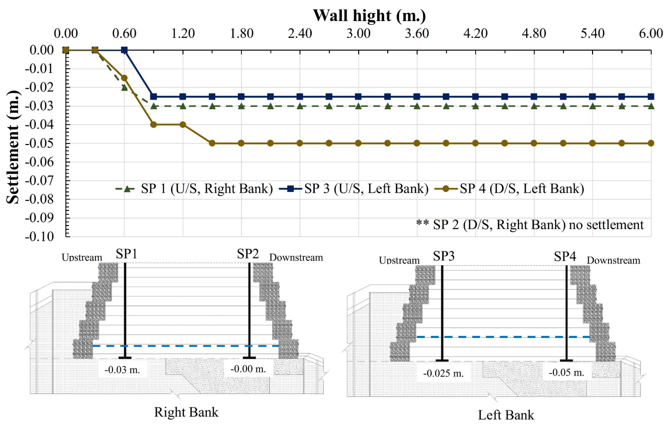
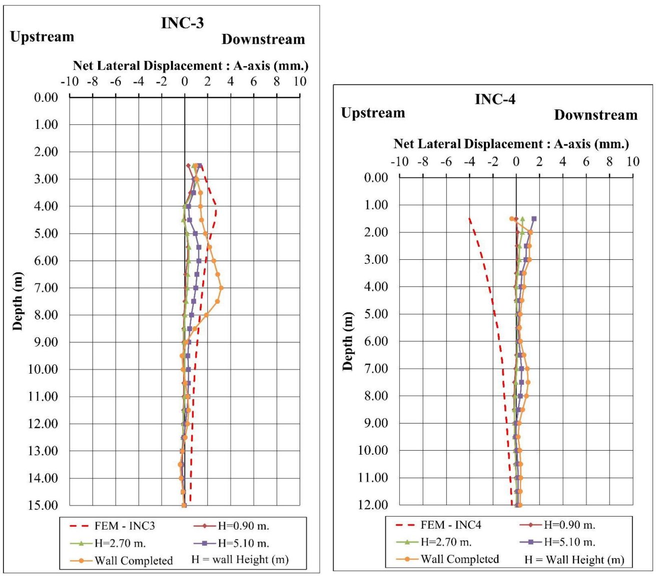


Figure 21 Vertical displacement of foundation of new dam crest (Settlement Plate)

6.2 Lateral Deformation

The lateral deformation of the new dam crest with MSEW obtained from field measurements using inclinometers according to the height of the wall construction has been compared with the FEA results at the end of the construction. Figure 22a shows the net lateral displacement obtained from upstream inclinometer (INC3) compared with the FEA results at the end of the construction. The lateral displacement obtained from field measurements and FE were consistent; it has very lowest deformation and was bent to downstream side. The maximum measured lateral displacement was 3 mm at depth of 7 m depth and the maximum lateral displacement obtained from FEA was 2.7 mm at 4 m depth. Likewise, no lateral displacement occurred from 9.00 m depth. Figure 22b shows the net lateral displacement obtained from upstream inclinometer (INC4) compared with the FEA results at the end of the construction. The results from the field measurements showed a slightly bent to downstream side while the result from FEA was slightly bent to upstream side. The maximum measured lateral displacement was 1.23 mm. and the result from FEA was 5 mm. Since various construction activities (upstream soil compaction, temporary road construction and its use) were being performed at the location where inclinometer was installed, the differences occur due to the construction phase designation in the FEM. However, the measured and modeled lateral displacement was very low.



(a) (b)

Figure 22 Inclinometer reading and FEA results of test section at the left bank (a) INC-3, Upstream (b) INC-4, Downstream

6.3 Force and Strains in Reinforcement

The axial force in the reinforcement was measured using vibrating wire rebar strain meter bar. The comparison between FEA and axial force obtained from field measurement distribution along the rebar length and are in good agreement (Figure 23). At the reinforcement in layer 1 & 3 at downstream sides, the tensile force at the location behind the gabion was low and had increased as it moved further away. The maximum measured tensile force was 1,086 kg at the location behind the gabion facing 2.60 m. The tensile development during the construction is as shown in Figure 24a. It shows that the tensile force increases with increasing wall height. Meanwhile, the tensile force measured on the upstream side was very low (layer 1, 3 and 5) and the axial force in rebar reinforcement of layer 1 was compressive. The compaction of the clay between MSEW facing and steel sheet pile on the upstream side results in lateral earth pressure. The tensile force developed during the construction is shown in Figure 24b. The clay was not compacted at the beginning of the construction; therefore, the tensile force increases with the wall height and is consistent at the downstream side. The tensile force decreases and is converted to the compressive force as the height of wall increases.

However, the tensile stress in rebar from both field measurements and FEA was low as compared to the yield strength of rebar. The tensile strain in the geogrid reinforcements was measured using ultrahigh-elongation foil strain gauges. The comparison between FEA and field measurements tensile strain distribution along the geogrid length are in good agreement and is shown in Figure 25. The strain measured in all the positions were very low. The maximum field measured strain was 180 micro strain or 0.018% and the maximum strain obtained from FEA was 350 micro strain or 0.035%, which was very low as compared to the elongation at break of geogrid obtained from laboratory tests, which was 3.49%. It shows that the geogrid does not elongate according to the behavior of reinforced soil wall.

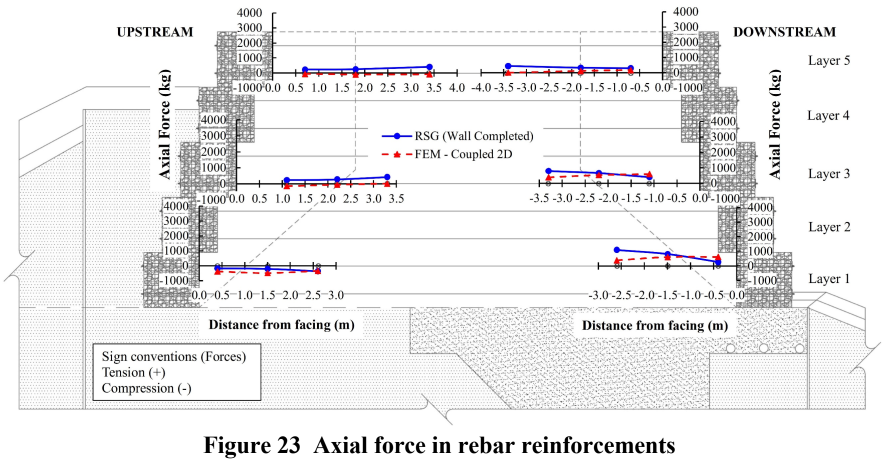
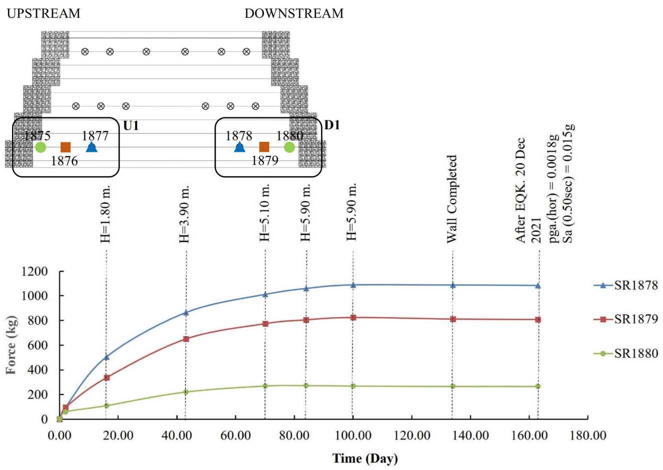
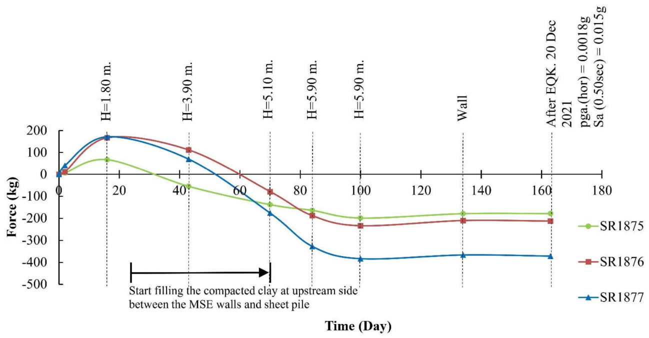


Figure 23 Axial force in rebar reinforcements



(a)



(b)

Figure 24 Axial force in rebar reinforcements with construction stage (Time, Wall height (H)) (a) downstream side, D1 (b) upstream side, U1

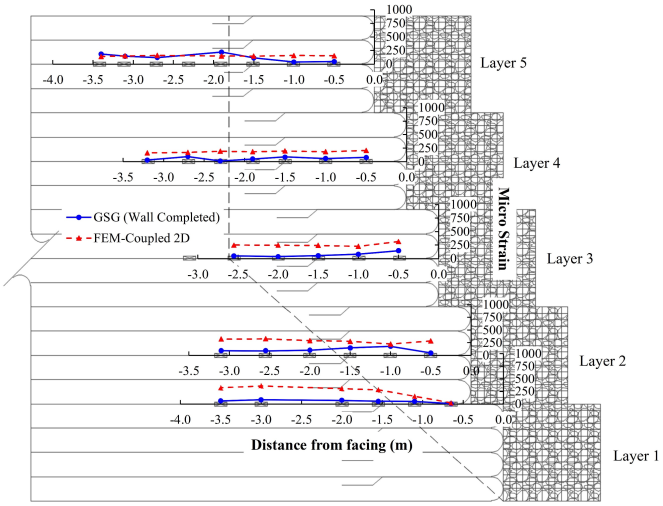
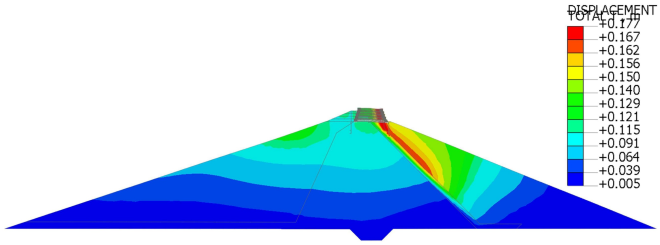


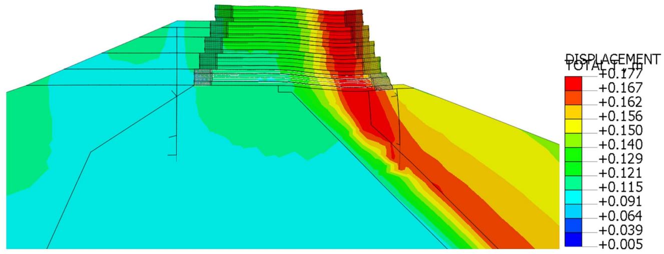
Figure 25 Tensile Strain in Geogrid

6.4 Stability of MSE Wall for Dam Crest Rehabilitation

Stress-seepage coupled analysis in FEM has revealed that the behavior of MSEW and new dam crest during construction and operation are consistent with the field measurement data. In this research, slope stability analysis was performed using strength reduction method (SRM) which was the continuation of the Stress-seepage coupled analysis. Figure 26 shows the total displacement of earth dam from the slope stability analysis (SRM). The failure plane occurs on downstream side along the filter passing through the bottom of the sheet pile, the factors of safety (F.S.) of MSEW for dam crest rehabilitation was 3.10. Figure 27 shows the operation of Mae Suai Dam in year 2021 after the completion of rehabilitation process.



(a)



(b)

Figure 26 Total displacement of earth dam from the slope stability analysis (SRM)

Figure 27 Mae Suai Dam in operation again since 2021.

7. CONCLUSIONS

A 6-m-high reinforced earth embankment for dam crest rehabilitation was designed and constructed at Mae Suai dam, Chiangrai province, Thailand. Welding mesh gabion was used as the facing on both sides; the polymetric geogrid and rebar were used as reinforcements. Furthermore, the MSE wall was placed on the original earth dam crest, steel sheet pile was installed at upstream and downstream side to prevent leakage and control the settlement of the original earth dam crest. The instruments were installed at various test sections. The field measurement data was verified with 2D FEM using MIDAS GTS for Stress-seepage coupled analyses. Furthermore, slope stability analysis was performed using strength reduction method (SRM) to determine the F.S. of the dam crest rehabilitation. From the analysis results, the factor of safety of the new dam crest was 3.10. The behavior of restraint back-to-back MSEW for dam crest rehabilitation is as shown below:

- Vertical displacement

The settlement of the foundation obtained from FEA was very low as compared with the height of the dam or MSEW because the foundation of the MSEW was an earth dam that used to support the weight of the old dam crest, which was heavier than the new dam crest. Likewise, the field measurement data also shows the settlement behavior that was an immediate settlement caused by construction conditions.

- Lateral deformation

The lateral displacement obtained from field measurements and FE were consistent i.e., it has very low deformation. It shows that the good performance of sheet pile was able to control lateral deformation which had occurred due to the new dam crest.

- Force and strains in reinforcement

The measured and FEA in the rebar and geogrid reinforcement were in good agreement. Tensile force in rebar on downstream side was higher than upstream side and the maximum tensile force was located at the bottom layer. The tensile force decreased with increase in height. Meanwhile, the tensile force measured on the upstream side was very low; the axial force in rebar reinforcement of the bottom layer was a compressive caused by the lateral earth pressure of compacted clay on the upstream side. These measured forces were found to be continually changing due to deformation of foundation, external stimuli and construction factors. Likewise, the strain measured in all positions of geogrid reinforcement was very low which questions the performance of geogrid. Combining steel reinforcement (high stiffness) with geogrid reinforcement (low stiffness) was redundant. Most of the lateral stresses are resisted by the former than the later as obtained in the results. It can be concluded that in a reinforced soil wall that uses two or more types of reinforcing materials, tensile force is developed in higher stiffness material. However, the tensile stress in rebar from both field measurements and FEA was low compared to the yield strength of rebar. It can therefore be concluded that the dam crest rehabilitation was a highly successful.

8. ACKNOWLEDGMENTS

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