The Use of Deep Soil Mixing as Channel Slope Stabilization Method – US Army Corps of Engineers Bechara Middle Section Project Case Study

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Abstract *— This project describes and validates the design of a slope stabilization technique for the Bechara Middle Section (BMS) channel located in the Bechara Industrial Area, San Juan, Puerto Rico. The BMS project consisted of approximately 720-foot earth open-channel and is a subcomponent of the Rio Puerto Nuevo Flood Control Project constructed by the US Army Corps of Engineers. Pre-construction soil conditions in the site consisted of soft clays and organic material that precluded the excavation of the proposed channel geometry. The soil slope stabilization design option for the BMS was two continuous soil treated zones, A and B, by means of improving the existing soft soil conditions with the technology of Deep Soil Mixing. The slope stability of the channel was analyzed by modeling the improved soil in GeoStudio-Slope/W and was found to be in compliance with the required safety design factors.*

Key Terms *— Deep soil mixing, earthquake pseudo-static analysis, open channel, slope stability*

INTRODUCTION

The Bechara Middle Section project is located in the Bechara Industrial Park, near the Puerto Nuevo Complex Port, in San Juan Puerto Rico. The project was constructed between 2012 and 2016 as part of the Rio Puerto Nuevo Flood Control Project by the US Army Corps of Engineers (USACE) to reduce the chances of flooding in the Bechara Sector. The work of this project includes the construction of an approximately 720-foot earth open-channel between the end of an existing concrete box culvert and Kennedy Avenue, using the Deep Soil Mixing (DSM) technology as soil stabilization method to improve subsurface conditions. Figure 1 shows the location map of the project. The scope of this case study is to conduct a geotechnical analysis using slope stability methods and validate the results of using DSM technique as an option for slope stabilization.

Figure 1
Project Location

BACKGROUND

The Bechara Middle Section (BMS) Channel project is a subcomponent of Río Puerto Nuevo (RPN) Flood Control Project of the USACE. The purpose of the authorized RPN Flood Control Project is to protect lives and property from damages attributable to a 1% exceedance probability flood along the River and its tributaries. This level of protection is commonly called "100-year" flood protection [1].

Prior to 1950s the Río Puerto Nuevo originally flowed into San Juan Bay through the BMS project vicinity and the river's mouth was located at the Puerto Rico Port Authority (PRPA) docks. In the 1950s the river's mouth and lowermost ¾ mile of channel were re-routed to the east, to empty into Martín Peña Channel. In the early 1960s, after the river had been diverted, the PRPA began to build the Puerto Nuevo Port Complex, and USACE dredged the new Puerto Nuevo Navigation Channel in San Juan Harbor to serve these docks [1].

The creation of the Puerto Nuevo port area and diversion of the River stimulated public, commercial, and industrial development along John F. Kennedy Avenue, and the avenue became a major arterial road for port traffic and commuters. The Bechara Industrial Area then became part of this commercial/industrial development. The new port was built over fill deposited into the area north of Kennedy Avenue (formerly all mangrove swamp). This fill effectively "plugged" the lower end of the natural Puerto Nuevo River drainage and did not provide an alternate outlet for drainage north of Margarita Creek.

The construction of the Bechara Middle Section provided the drainage infrastructure required by the developed Bechara Industrial Park area by connecting the existing industrial channel south of the work area with the existing concrete box culvert located north. These were both built in previous phases of the project. The lack of this connection often caused high water stages and flood on the industrial/commercial area upstream.

SITE DESCRIPTION

The pre-construction site conditions of the project studied, which is located between stations STA 22+60 to STA 29+80 of Bechara Industrial Area (BIA) Channel, consisted of manmade fill covered by mangroves and vegetation. The site is surrounded by the PRPA Puerto Nuevo Port Complex to the north and east. At STA 22+60 the project is connected to a concrete box culvert that runs under the Puerto Nuevo Port area, which is downstream section of the BIA Channel. To the south, at STA 29+80 the site is connected to the upstream section of the BIA Channel at the Kennedy Avenue Bridge (westbound) and a local drainage ditch to the southeast that runs parallel to the Kennedy Avenue. The upstream and downstream sections of the project were previously constructed between 2002 and 2010. Ground surface elevation along the footprint of the proposed BMS channel varied between elevations -10.0 feet and 13.0 feet using National Geodetic Vertical Datum (NGVD). Figure 2 shows the aerial view of the project site and the scope of work details.

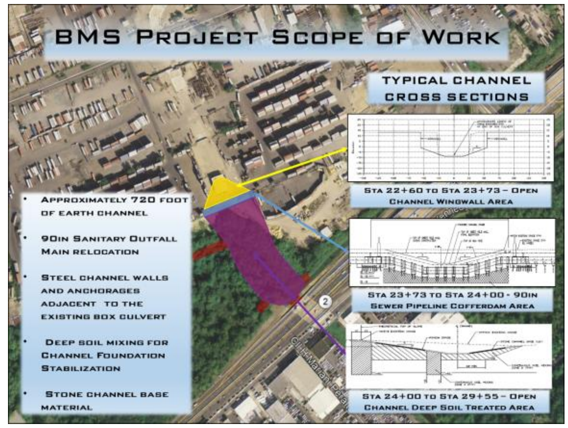

Figure 2
Aerial View of Site and Project Scope of Work

SOIL EXPLORATION AND LABORATORY TESTING

Four disturbed borings were selected from a total of thirteen existing soil borings available along the entire Bechara Industrial Park Channel to obtain design soil parameters for the analysis. Borings CB-BC99-106, CB-BC00-107, CB-BC00-C1, and CB-BC00-C2 were selected based on their location within the proposed footprint of the BMS channel between STA 21+00 to STA 29+80. Their penetration depth vary from 40.5 to 110.0 feet below ground surface. Figure 2 presents a map of the vicinity and the soil boring location information.

Soil properties data was collected from [2], which has all laboratory investigation conducted on soil samples retrieved from the four borings. These samples were retrieved using split barrel sampler via Standard Penetration (SPT) Method (ASTM D1586) [3].

The soil design parameters for the slope stability analysis of this study were based on the interpretation of N-values test results of the penetration resistance of the soil and engineering judgment using typical N-value correlations.

Figure 3 shows the boring location map of the soil exploration.

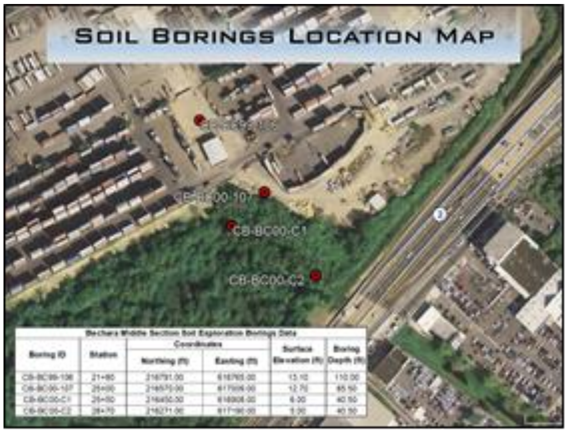

Figure 3
Location Map of the Soil Borings

SUBSURFACE CONDITIONS

Subsurface conditions in the location of the project include manmade fill consisting predominately of silty sand (SM) with layers of silt (ML) and poorly graded sand (SP) from elevation (El.) 13.10 ft to El. -8.0 ft. Swamp deposits mainly consisting of very soft to soft organic fat clays (OH) and organic silts (OL) interbedded layers of peat (PT) and lenses of shell fragments are generally encountered from El. -8.0 ft to El. -37 ft below the manmade fill.

Below the swamp deposits, material consisting predominately of interbedded medium to very stiff fat clays (CH), lean clays (CL) and silts (ML) extend to elevations of approximately El. -57 ft. Below elevations of approximately El. -57 ft to El. -64 ft interbedded stiff to hard poorly graded sand (SP), clayey sand (SC) and silty sand (SM) with limestone fragments are encountered. Below El. -64 ft to El. -97 ft, conditions consist predominately of weathered limestone with fine to coarse grain sizes, interbedded layers of very stiff to hard silt (ML) and stiff to very stiff lean clay (CL). Figure 4 depicts the four borings and the generalized soil profile created for the study.

Soil Parameter Selection

Design soil parameters for the stability analysis were selected using the data collected from the four soil borings. For the shear strength, the SPT blow count number (N) was used in conjunction with correlation to estimate the cohesion of clayey material and the angle of friction of granular soils. Since the N-values obtained from SPT samples were not corrected, the following empirical equation established by Skempton [4] was used:

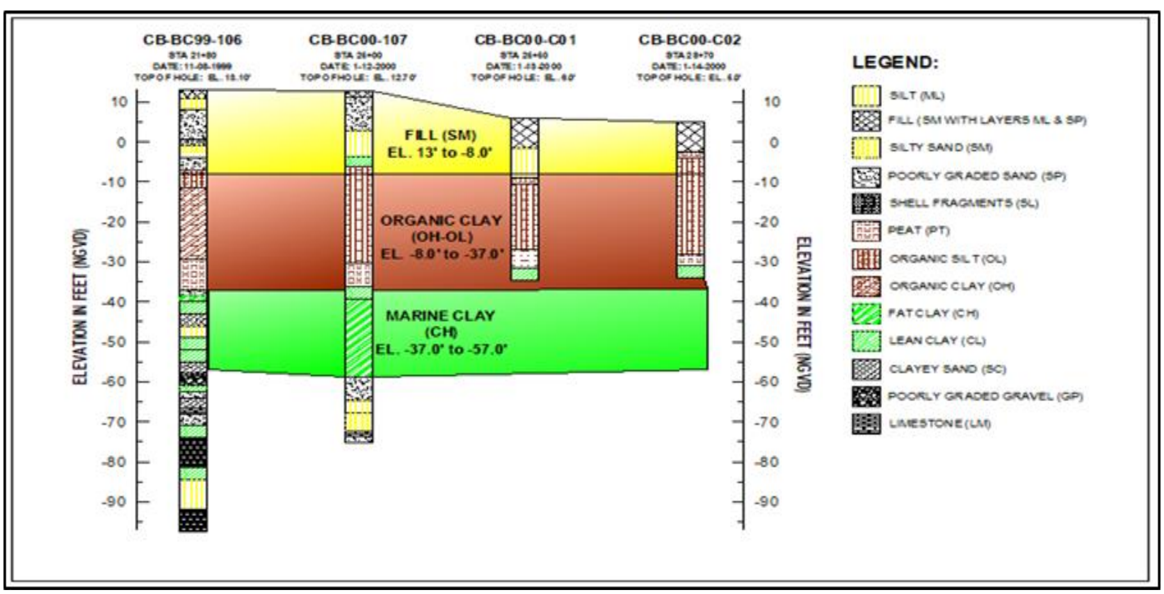

Figure 4
Boring Logs and Generalized Soil Profile

$$N_{60} = \frac{E_m \times C_B \times C_S \times C_R}{0.60} \qquad (1)$$

where:

- N_{60} = SPT N-value corrected for field procedures
- E_m = hammer efficiency (E_m = 0.575 assumed)
- C_B = borehole diameter correction (C_B = 1.0 assumed)
- C_S = sampler correction (C_S = 1.0 assumed)
- C_R = rod length correction (C_R = 0.85 assumed)
- N = measured SPT N-Value

Once the N_{60} values were obtained, a representative blow count for each layer of the generalized soil profile for the study was calculated using arithmetic average. Figure 5 presents a plot of the N_{60} values for each of the four borings and variation with depth. Using N-values correlations recommended in [5], the shear strength for each layer was determined.

Recommendations in [6] were also used to determine baseline values of cohesion for fine grain soils and angle of friction for granular material for analyses of unconsolidated-undrained conditions (Q-Case) and consolidated-drained conditions (S-Case). Tables 1 and 2 present the soil parameters used for the Q-Case and S-Case in the stability analysis of pre-construction conditions, respectively.

Table 1

Design Parameter for Undrained Analysis (Q-Case) of Original Soil Conditions

Depth (ft)	γ_{wet} (pcf)	γ_d (pcf)	Soil Type	Cohesion (psf)	Angle of Friction ϕ'	Soil Shear Strength $\tau = c + \sigma' \tan(\phi)$ (psf)
13 to -8.0	119	96.7	SM (Fill)	0	30	0 (top) 1129 (bottom)
-8.0 to -37.0	91	50.6	OL-OH (Organic Clay)	250	0	250 (top) 250 (bottom)
-37.0 to -57.0	109	76.2	CH (Marine Clay)	950 950	0 0	950 (top) 950 (bottom)

Table 2

Design Parameter for Drained Analysis (S-Case) of Original Soil Conditions

Depth (ft)	γ_{wet} (pcf)	γ_d (pcf)	Soil Type	Cohesion (psf)	Angle of Friction ϕ'	Soil Shear Strength $\tau = c + \sigma' \tan(\phi)$ (psf)
13 to -8.0	119	96.7	SM (Fill)	0	30	0 (top) 1129 (bottom)
-8.0 to -37.0	91	50.6	OL-OH (Organic Clay)	0	23	830 (top) 1407 (bottom)
-37.0 to -57.0	109	76.2	CH (Marine Clay)	0	23	1407 (top) 2055 (bottom)

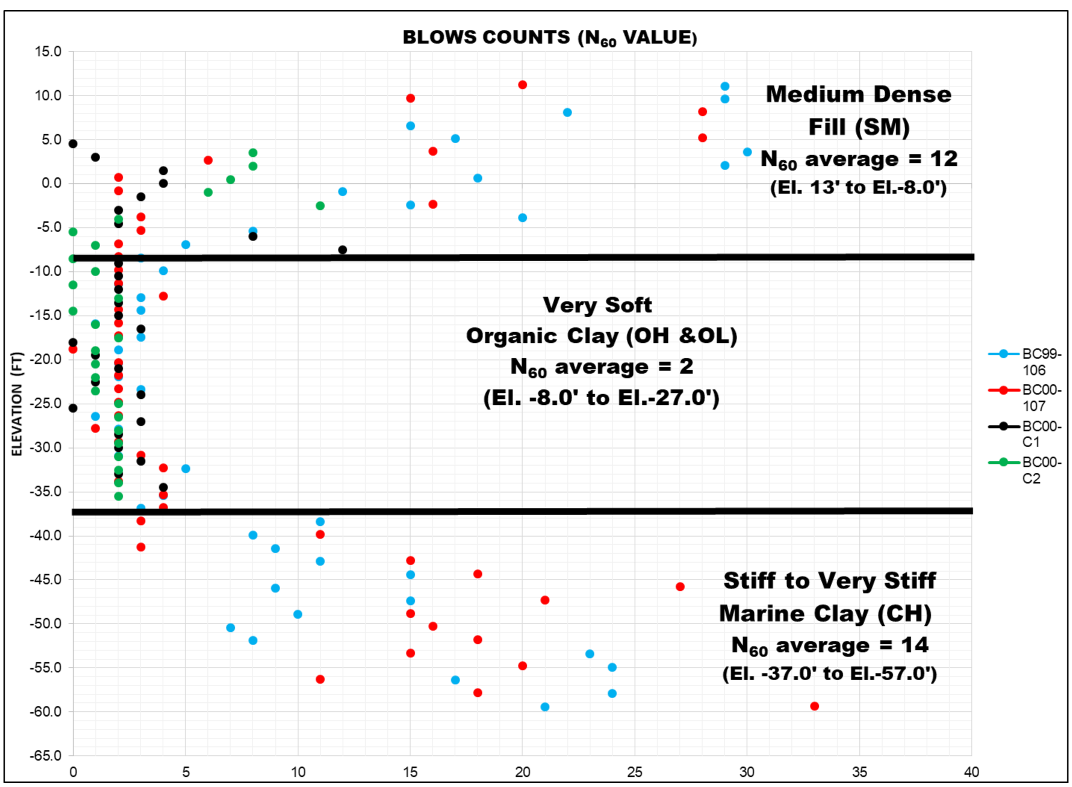

Figure 5

Variation of SPT Corrected Blow Counts "N_{60}" with Depth

Geotechnical Design Criteria

The BMS Channel geometrical design consisted of a trapezoidal shape open-channel with 1V:5H side slopes, top channel width of 250 ft, and bottom channel width of 25 ft at a depth of El -9.0 ft. The channel slopes and bottom are covered by 1.50 ft stone base that serve as scouring protection.

The geotechnical design criteria used for this study is shown in Table 3. The factor of safety (FoS) requirements shown in the table were the minimum slope stability requirements of [7] using circular slip surfaces. [8] was used for the criteria of stability analysis during earthquake events. The different analysis cases were determined based on the engineering judgment of the intended purpose of the channel, its conditions during the phase of construction and final operational stage.

Table 3

Slope Stability Design Criteria

Slope Stability Analysis Cases	Minimum Factor of Safety Required
Final Construction Stage with Excavation Open at El -9.0' (Q-Case)	1.5
Final Construction Stage with Excavation Open at El -9.0' (S-Case)	1.3
Operational Stage with Channel Water at El 1.0' (Q-Case)	1.5
Operational Stage-Drawdown with Channel Water at El -2.0' (S-Case)	1.3
Operational Stage with Channel Water at El 1.0' (Q-Case) - Earthquake Event	1.0

The cross section used for the analysis was the most critical of the channel located at STA 24+00, with both tops of bank ground elevation of 13.0 feet and total excavation height of 22 ft.

Existing Conditions Analysis

The BMS channel was modeled in GeoStudio-Slope/W 2018-R2 Student Version software using the existing soil conditions shown in Tables 1 and 2, and its proposed geometrical layout to determine the FoS under the initial conditions. Analyses were performed using circular slip failure methods of Ordinary Method of Slices (OMS), Spencer, Morgenstern-Price, Bishop and Janbu. Failure mechanisms are based on force-moment equilibrium considering normal and shear forces between slices.

Based on the available survey and soil data of the site, the analyses on current conditions showed that if the BMS channel was constructed in such soil conditions, the slopes were going to be unstable. Table 4 presents the results summary of the stability cases analyzed and their respectively factors of safety. Figure 6 presents the model of the most critical case analyzed.

Table 4

Factor of Safety of Existing Soil Conditions

Slope Stability Analysis Cases	Minimum Factor of Safety Required	OMS	Spencer	Morgen stern-Price	Bishop	Janbu
Final Construction Stage with Excavation Open at El -9.0' (Q-Case)	1.5	0.624	0.625	0.631	0.636	0.555
Final Construction Stage with Excavation Open at El -9.0' (S-Case)	1.3	0.821	0.924	0.925	0.869	0.786
Operational Stage with Channel Water at El 1.0' (Q-Case)	1.5	0.764	0.777	0.784	0.789	0.692
Operational Stage-Drawdown with Channel Water at El -2.0' (S-Case)	1.3	1.016	1.090	1.090	1.103	0.984

Design Solution - Deep Soil Mixing

Given the poor soil conditions of the site, the option of deep soil mixing (DSM) method was proposed as slope stabilization. The DSM procedure employs stabilizer admixtures such as cement, slag, and other pozzolanic materials to improve soil conditions and provide ground stabilization. These stabilizers agents are blended with the natural soil by mixing equipment that delivers the agents in dry powder form (dry method) or slurry form (wet method). The equipment usually consists of multiple-axis or track vehicle with vertical rotating shafts that have overlapping mixing paddles to create walls of overlapping soil mix columns.

The DSM design for channel stabilization consisted of continuous soil treatment areas divided in two zones, A and B, between STA 24+00 to STA 29+55. Table 5 presents the DSM parameters used in soil treatment operations, where 10,800 psf was the minimum compressive strength required for DSM design after soil was treated [2]. Figure 2 contains a channel cross-section view depicting zones A and B. As part of the quality control program, a series of DSM test columns were constructed onsite to validate the soil-cement mix design. Soil samples were retrieved from these columns and unconfined compressive strength results were between 33,120 psf to 64,800 psf for Zone A, and 31,680 psf to 94,480 psf for Zone B.

Table 5

Deep Soil Mixing Zone Treatment Parameters

Properties	Specified Values Zone A	Specified Values Zone B
Column diameter	5.5 ft	5.5 ft
Column Length	varies (12ft to 58 ft)	varies (9.5ft to 28ft)
Bottom of Treatment Elevation	-45 ft	-20 ft
Column Overlap Treatment	continous	continous
Water/Cement Ratio	1.50	1.25
Number of Mixer Blades	6	6
Specified Unconfined Compression Strength of Soil Mixed (psf)	18,000	10,800
Curing Time	28 days	28 days

Design Solution Slope Stability Analysis

Slope stability analyses were performed evaluating failure surfaces beneath and through the DSM zones of both banks' slopes (east and west) using the same cases analyzed with original soil conditions. DSM zone was modeled as a single soil treated area with cohesion of 10,800 psf due to Slope/W Student Version limitations. However, the approach taken was a conservative one using a lower soil shear strength than those obtained from test columns for both DSM zones. Other Slope/W Student Version limitations forced use of a combined layer of the Fill (SM) and Organic Clay (OH-OL) to model the soil treated conditions. Tables 6 and 7 present the soil parameters used for the Q-Case and S-Case in the stability analysis of soil treated conditions, respectively.

Table 6

Design Parameters for Undrained Analysis (Q-Case) with Deep Soil Mixing "DSM" Cement Treatment

Depth (ft)	γ_{wet} (pcf)	γ_d (pcf)	Soil Type	Cohesion (psf)	Angle of Friction ϕ'	Soil Shear Strength $\tau = c + \sigma' \tan(\phi)$ (psf)
Varies (13.0 to -20.0 or -45.0)	119	96.7	Deep Soil Treated Zones A & B	10,800	0	10,800
13.0 to -37.0	119	66.1	Natural Combined Soil (OL-SM)	250	0	250
-37.0 to -57.0	109	76.2	CH (Marine Clay)	950	0	950

Table 7

Design Parameters for Drained Analysis (S-Case) with Deep Soil Mixing "DSM" Cement Treatment

Depth (ft)	γ_{wet} (pcf)	γ_d (pcf)	Soil Type	Cohesion (psf)	Angle of Friction ϕ'	Soil Shear Strength $\tau = c + \sigma' \tan(\phi)$ (psf)
Varies (13.0 to -20.0 or -45.0)	119	96.7	Deep Soil Treated Zones A & B	10,800	0	10,800
13.0 to -37.0	119	66.1	Natural Combined Soil (OL-SM)	0	23	0 (top) 1,585 (bottom)
-37.0 to -57.0	109	76.2	CH (Marine Clay)	0	23	1,585 (top) 2,232 (bottom)

The global slope stability was controlled by deep-seated failures beneath the DSM zones for all cases analyzed. The west bank slope resulted with the lowest factors of safety of the channel. Table 8 presents the results summary of the stability cases analyzed and their respectively factors of safety. Figure 7 presents the model of the most critical case analyzed.

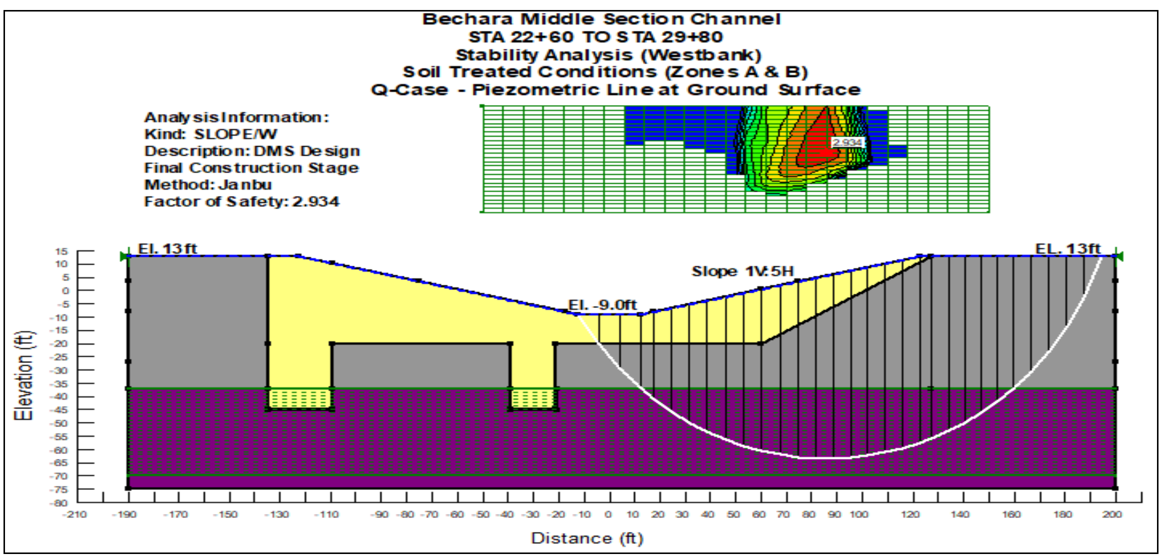

Figure 7

Janbu Model in Slope/W – Final Construction Stage with Excavation Open at El -9.0' with Soil Treated Conditions

Table 8

Factor of Safety of Soil Treated Conditions

Slope Stability Analysis Cases	Minimum Factor of Safety Required	OMS	Spencer	Morgen stern-Price	Bishop	Janbu
Final Construction Stage with Excavation Open at El -9.0' (Q-Case) West Bank	1.5	2.954	2.986	2.986	2.986	2.934
Final Construction Stage with Excavation Open at El -9.0' (S-Case) West Bank	1.3	3.233	3.699	3.707	3.707	3.524
Operational Stage with Channel Water at El 1.0' (Q-Case) West Bank	1.5	3.674	3.704	3.704	3.704	3.627
Operational Stage-Drawdown with Channel Water at El -2.0' (S-Case) West Bank	1.3	4.028	4.753	4.760	4.761	4.509
Final Construction Stage with Excavation Open at El -9.0' (Q-Case) East Bank	1.5	2.960	3.028	3.028	3.028	2.999
Final Construction Stage with Excavation Open at El -9.0' (S-Case) East Bank	1.3	3.244	3.753	3.762	3.760	3.564
Operational Stage with Channel Water at El 1.0' (Q-Case) East Bank	1.5	3.681	3.778	3.778	3.741	3.693
Operational Stage-Drawdown with Channel Water at El -2.0' (S-Case) East Bank	1.3	4.034	4.806	4.813	4.813	4.552

Slope Stability Analysis: Earthquake Event

Due to the fact that the project is located in an earthquake-prone location, a slope stability analysis was conducted using the pseudo-static analysis method. The pseudo-static method offers the simplest approach for evaluating the stability of a slope in an earthquake region. The limit equilibrium method is modified to include horizontal and vertical static seismic forces that are used to simulate inertia forces due to ground accelerations in an earthquake [8]. For this analysis, the critical slip surface obtained from the Ordinary Method of Slices (OMS) analysis for the final operational case of the channel was selected to obtain the soil mass parameters. Figure 8 shows the variation of the factor of safety with increase of horizontal acceleration (k_h). The vertical acceleration coefficient (k_v) was assumed to be zero (0), as recommended in [8].

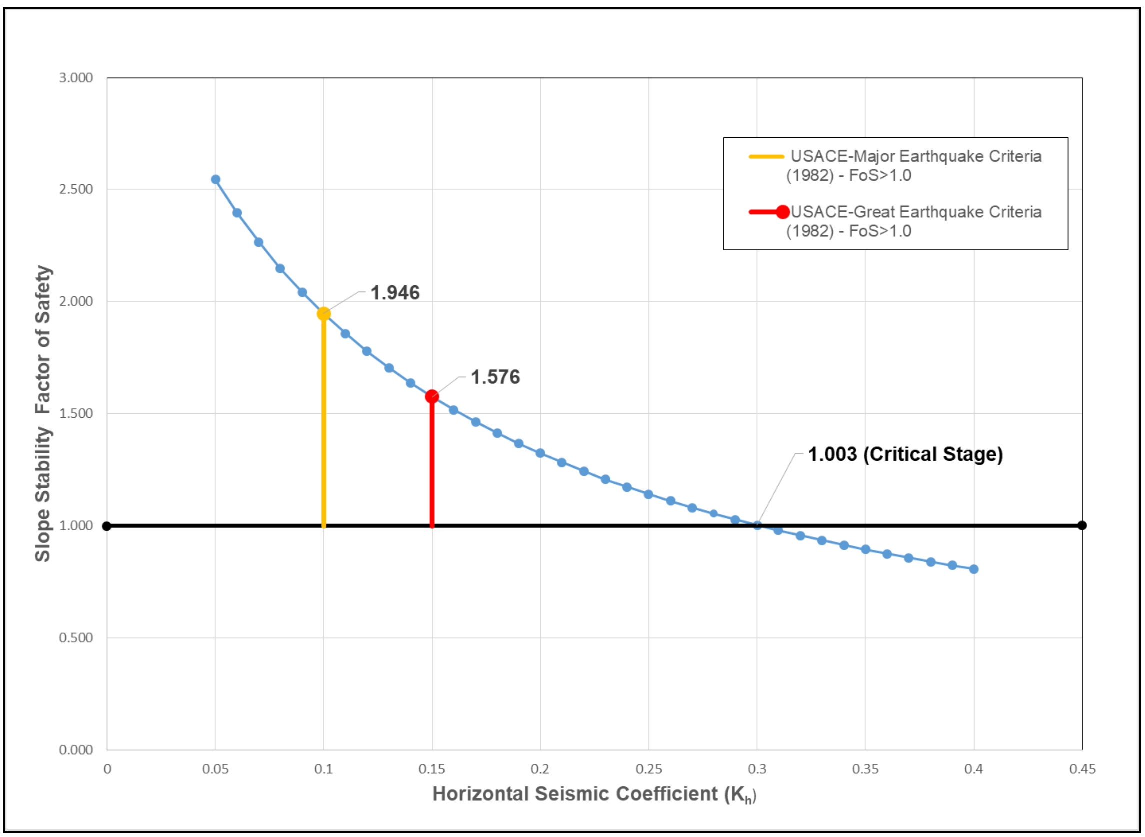

Figure 8

Variation of Factor of Safety with Horizontal Seismic Coefficient (k_h) for Final Operational Stage – West Bank

CONCLUSION

The original subsurface conditions of the BMS channel did not have the required soil shear strength capacity to meet safe USACE slope stability requirements for the flood control project. The results of the slope stability analyses using original soil conditions shown in Table 4 proved that the proposed geometry of the channel couldn't be achieved to the required depth of El. -9.0 ft, even during its construction phase. The design parameter presented in Table 5 used for the deep soil mixing technique was validated through the actual slope stability analysis of the channel. These soil treated conditions were modeled using shear strength parameters that were 40% lower than the actual values obtained from field test program and results were above the minimum factor of safety criteria. Even for the seismic conditions, the pseudo-static analysis presented in Figure 8 suggests that the DSM option can meet the required criteria during major earthquake and great earthquake events.

Based on this case study, the Deep Soil Mixing technique presents a feasible slope stability improvement solution for the channel with poor subsurface conditions. Figure 18 shows an aerial photo of the project before and after construction.

FUTURE WORK

The following are recommendations for future work of the Deep Soil Mix as Stabilization technique on this project:

- Conduct settlement evaluation of the deep soil treated mass to determine its behavior and long term effects on the flood control channel operation and maintenance.

Figure 18

Google Earth Aerial Image of the Project before and after Construction

- The use of other slope stability software such as UTEXAS and USACE-Stability with Uplift to analyze block mode failures and compare them with results of circular mode failures.
- The use of other slope stability software such as PLAXIS 3D to perform seismic analysis.
- One of the main features of work of the BMS Project was the construction of a 365-ft-long steel sheet pile cofferdam with depth varying from 21 to 39 ft for the relocation of the 90-inch-diameter sewer force main pipeline. It will be interesting to conduct a study of using Deep Soil Mix columns as temporary retaining structure technique and determine its feasibility to compare it with steel sheet pile wall technique.

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