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Assessment of Bridge Protection Schemes against Scouring and Bed Degradation at the Pasig-Potrero River

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ABSTRACT

The portion of the Pasig-Potrero River being traversed by the SCTEx (Subic-Clark-Tarlac Expressway) bridge constructed in the mid-2000's is found 17km east of the crater of Mt. Pinatubo. The riverbed is mostly overlain with lahar, which originated from Mt. Pinatubo when it erupted in 1991. Several rainfall events, especially typhoons and rainfall induced by the *Habagat* (Southwest monsoon), caused massive sediment transport which led to bank erosion and eventual exposure of the bridge pier pile caps. In order to mitigate bed degradation and further exposure of the pile caps, pier protection structures, as well as a ground sill at the downstream side of the bridge, were recommended. The hydraulic behavior and sediment transport within the river were analyzed, and the corresponding results were used to establish the extent of the bridge foundation protection schemes. Non-structural measures such as regular monitoring of the pier protection scheme and level of sediment accumulation were also firmly recommended.

Keywords: bed degradation, scouring, lahar, ground sill, pier protection, bridge pier foundation

1. INTRODUCTION

The Pasig-Potrero Bridge is located along the SCTEx (Sublic-Clark-Tarlac Expressway), which is a 94-km expressway that connects the provinces of Zambales, Bataan, Pampanga and Tarlac in Central Luzon, Philippines. Based on observation, this portion of the

Pasig-Potrero River is prone to sediment transport, in the form of scouring and degradation, which are expected to be aggravated during strong rainfall events. The bridge is located at a bend of the Pasig-Potrero River approximately 17km from the crater of Mt. Pinatubo (Figure 1).
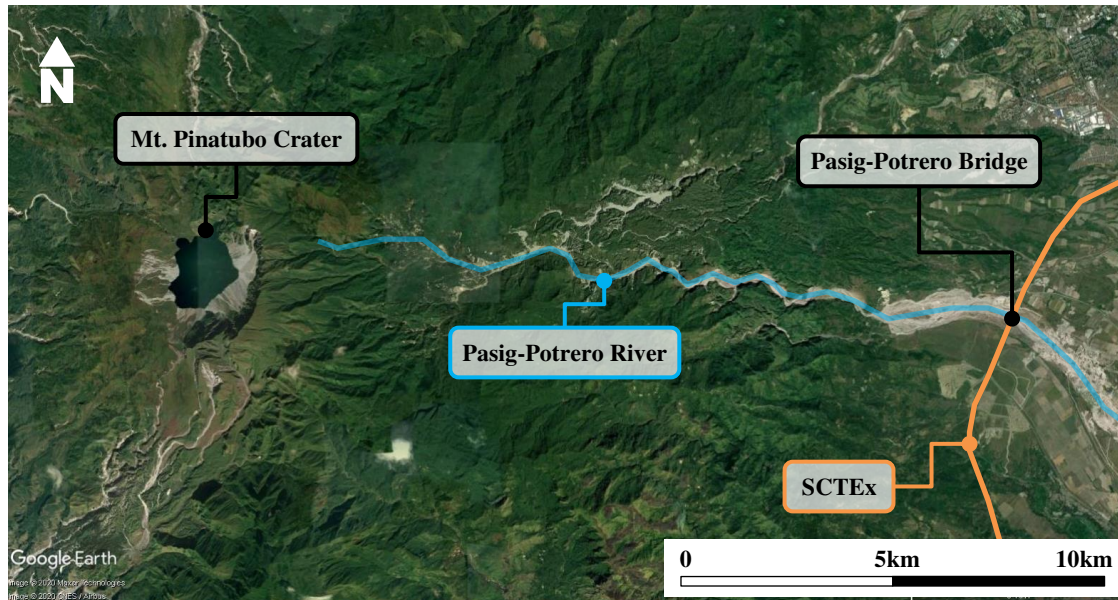

Figure 1. Location of the Pasig-Potrero Bridge (Google Earth)

The 1991 Mt. Pinatubo eruption caused up to 6km^3 of lahar deposition on the flanks of the volcano, filling adjacent river valleys, including areas within the Pasig-Potrero River Basin, with up to 200m of volcanic debris (Tuñgol, 2002). Since then, the riverbed has been overlain with lahars, and cases of lahar flow driven mostly by intense rainfall have continuously changed the river morphology. The Pasig-Potrero River is thus inherently predisposed to high sediment transport due to its bed material and steep slope coming from its headwater at the crater of Mt. Pinatubo. In 2013, *Typhoon Maring* intensified the rain induced by the southwest monsoon (locally called *Habagat,* which usually occurs from June to October every year), which resulted in strong river currents within the Pasig-Potrero River. The difference in velocity and shear stress across the bridge section of the river at that time has led to the erosion of the abutment at the outer bend and triggered the eventual failure of the approach embankment of the bridge.

Aside from bank erosion, bed degradation is also a problem caused by sediment transport. When the channel bed continues to lower as a result of more sediments leaving than entering, the river is in a condition called "degradation" (whereas "aggradation" is the opposite, wherein the channel rises in elevation due to accumulation of sediments). In contrast to bank erosion, which is locally triggered by lateral stresses within the river, bed degradation happens vertically throughout the profile and section of the river. Similar to bank erosion, bed degradation could also be triggered by strong flows within the river, such as during the *Habagat-*

influenced rainy season. As of 2018, due to strong rains brought by *Typhoon Ompong* in September, exposed pier pile caps have already been observed due to local scouring occurring with the bed degradation. A year after, comparing the elevations of the channel bed in March 2015 and September 2019, the maximum difference in elevation has then reached 9m. The bored piles at Piers 6, 7, and 8 have already been exposed (Figure 2).

Figure 2. Scouring at the Piers of the Pasig-Potrero Bridge. (Taken on September 2019)

Aside from the bridge piers, the abutments were also observed to show scouring. The piles of Pier 6 are already exposed, although parts of the pier are still embedded in the abutment, while Pier 11 is still within the abutment but shows scouring of the gabion bank protection (Figure 3). Based on these observations, mitigation for bank erosion shall be recommended for Piers 6 and 11.

Figure 3. Conditions at Piers 6 and 11 (Taken on September 2019)

2. METHODOLOGY

Data Collection. Initial site investigation was done to assess the existing conditions of the Pasig-Potrero Bridge. Data were culled from public data (i.e. government agencies), from the concession handling the expressway, and from first-hand field data collection. The topographic survey of the river was done last March 2019. Rainfall data were obtained from the PAGASA (Philippine Atmospheric, Geophysical and Astronomical Services Administration) Rainfall Station in Porac, Pampanga, which is approximately 7km from the project site.

Formulation of Recommendations. Based on the initial assessment, the short-term (Scour Protection) and long-term (Ground Sill) schemes for the bridge pier protection were determined. Scouring results when the flow of water carries away material from the bed or banks of streams. The rate of scour depends on the erosive power of the flow, erosion resistance of the material, and the sediments being transported into and out of a section. Local scouring usually occurs around piers, abutments, spurs, or embankments. In order to provide immediate protection to the piers against scouring, armors made of sheet piles were recommended. A ground sill at the downstream side of the bridge was also proposed as recommendation for the retention of the sediments, to encourage aggradation of the channel bed at the bridge. Hydraulic and geotechnical analyses were then done to verify the feasibility of the proposed schemes.

Hydrologic Analysis. Topographic data from NAMRIA (National Mapping and Resource Information Authority) and Google Earth were used to delineate the extent of the catchment of the Pasig-Potrero Bridge and its sub-catchments. The time of concentration (t_c), defined as the time for surface water to travel from the most distant point in the area to the outlet, was computed using the SCS (Soil Conservation Service) Lag Equation and was used as input to the hydrologic model. Rainfall intensity refers to the amount of rainfall in mm/hr (or in/hr) that falls within the catchment area, and is calculated from a previously determined Rainfall-Intensity-Duration-Frequency Curve equation obtained from PAGASA, with the time of concentration as the determining value. The 24-hr design rainfalls for 25-yr, 50-yr, and 100-yr return periods were determined. The 72-hr 100-yr return period rainfall event based on the probable maximum rainfall data (1050mm) was also considered to account for longer duration of rainfall such as that of *Typhoon Ompong* last September 2018. The value of the probable maximum rainfall was obtained from the report on Flood Control and Sabo Project prepared by JICA (Japan International Cooperation Agency) in 1978. This value was compared with the 72-hr rainfall value of *Typhoon Rita* in 1972 (734mm), corresponding to the "*Great Luzon Flood*", which is considered as the worst flooding in Central Luzon in recent history. The mentioned probable maximum rainfall was therefore accepted, with an additional multiplier of 1.1 to account for climate change projections (DPWH DGCS Volume 3, 2015). The resulting design hyetograph of this rainfall event, using the Alternating Block Method, is shown in Figure 5.
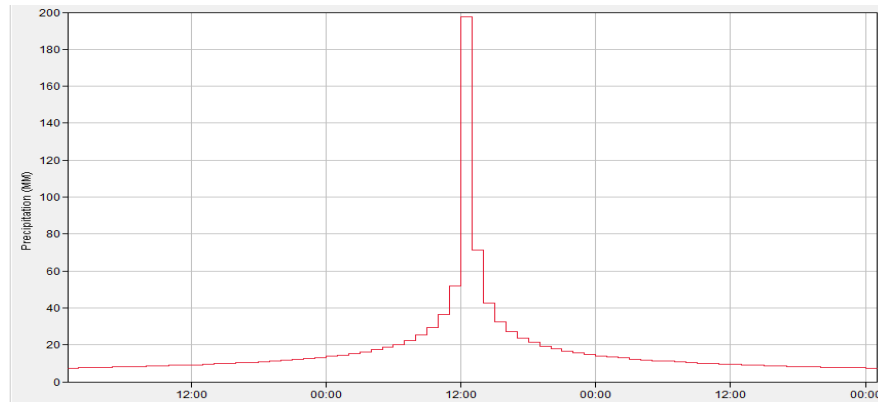
Figure 4. Design Hyetograph for 7-hr 100-yr Return Period Rainfall

The hydrologic model was done with the aid of HEC-HMS (Hydrologic Engineering Center – Hydrologic Modeling System). HEC-HMS is a software produced by the US Army Corps of Engineers that aims to simulate the hydrologic processes of watershed systems. For this study, the sub-basins and their reaches were modelled. The Pasig-Potrero Bridge was assumed as the sink wherein the floodwater from the entire catchment drains into. The hydrologic model is shown in Figure 6. Considering the catchment characteristics, losses, times of concentration, and reach routing, the peak discharges for the different design hyetographs were simulated in the HEC-HMS software. The resulting peak discharges are 234.24 m^3/s, 265.68 m^3/s, and 297.22 m^3/s, for the 24-hr 25-yr, 50-yr, and 100-yr return period rainfalls, respectively, and 780.93 m^3/s for the 72-hr probable maximum rainfall.
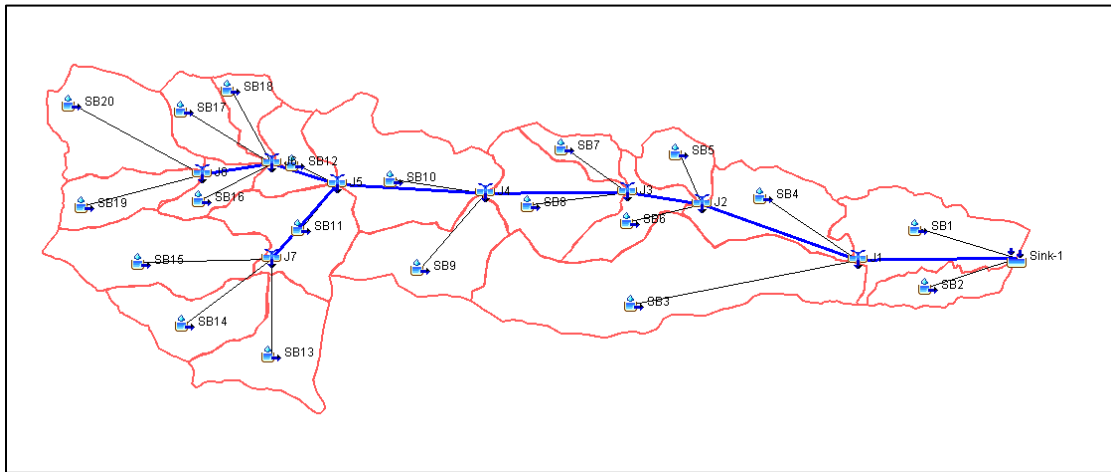

Figure 5. HEC-HMS Model of the Pasig-Potrero Catchment

Hydraulic Analysis. The Pasig-Potrero River was analyzed with the aid of HEC-RAS (Hydraulic Engineering Center River Analysis System). HEC-RAS is a public-domain program developed by the US Army Corps of Engineers that allows modeling of one-dimensional steady flow, unsteady flow, sediment transport/mobile bed computations, and water temperature

modeling. For this study, HEC-RAS was utilized to simulate one-dimensional steady flow on the waterway using the discharges obtained from the hydrologic calculations. The recommended bridge protection schemes were also included in the hydraulic model. The corresponding water levels and velocities for the different return periods were then used to assess the extent of the bridge protection schemes.
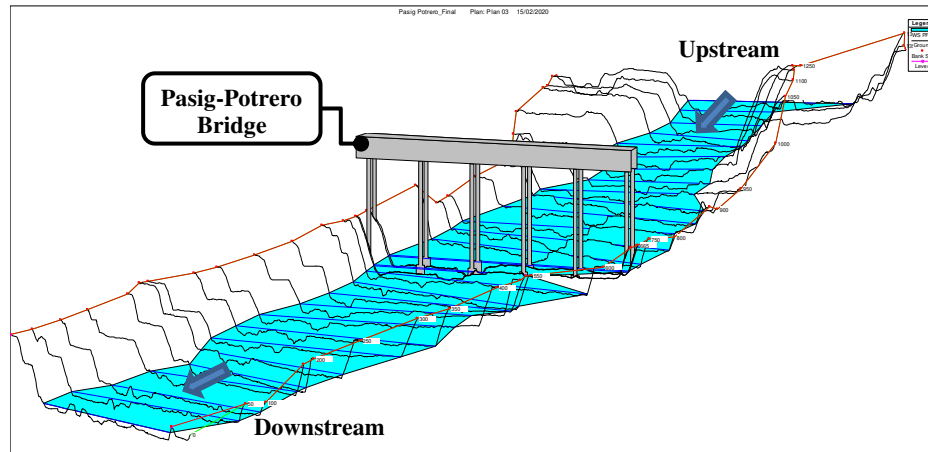

Figure 6. Hydraulic Model of the Pasig-Potrero River

Geotechnical Analysis. Geotechnical investigations and analysis were undertaken. At the identified locations, standard penetration tests (SPT) were conducted. Complementing the field activities is the laboratory testing of the samples obtained. Representative soil samples obtained during excavation were subjected to routine laboratory tests. The results of the field work and laboratory investigation were then used to establish the subsurface profile and the parameters for determining important geotechnical parameters. Based on the idealized subsurface layers, the soil was considered as medium dense to very dense sand. Geotechnical evaluation was then carried out to obtain the strength parameters necessary for the analysis and design of foundation protection.

3. PIER PROTECTION

As an immediate response to the exposure of the pier bored piles due to bed degradation, the piers were recommended to be protected with an enclosure. Among gabion, reinforced concrete, and steel sheet piles, the first two materials were deemed to be susceptible to scour and damage due to the strong flows in the river. Sheet piles were therefore selected for stronger resistance to flow. The existing piers shall be enclosed with anchored sheet piles (with one meter offset from the pile caps) and then backfilled for stability. This scheme considers backfill up to the level of the top of the pile cap, with sheet piles bended at the corners to minimize induced turbulence (Figure 7). This configuration was fixed, with the embedment length determined through the succeeding analyses.
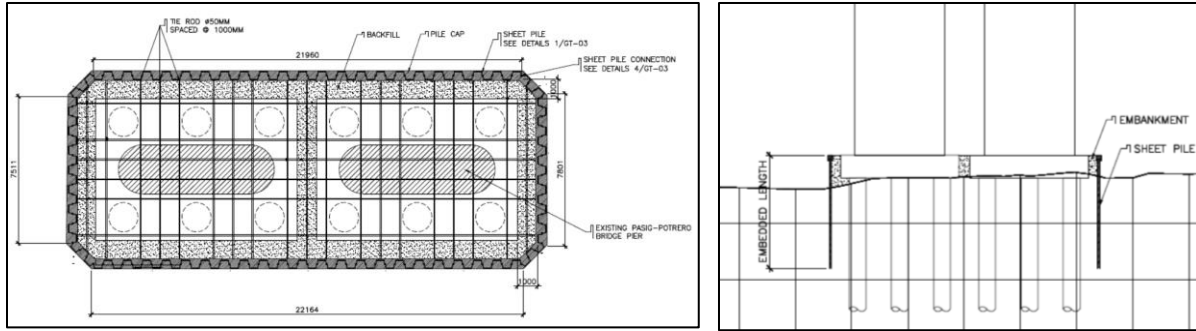
Figure 7. Typical Sheet Pile Protection Scheme

The scouring at the piers was incorporated in the hydraulic model using HEC-RAS. The Pasig-Potrero Bridge is located at Sta. 0+650 (Figure 8).
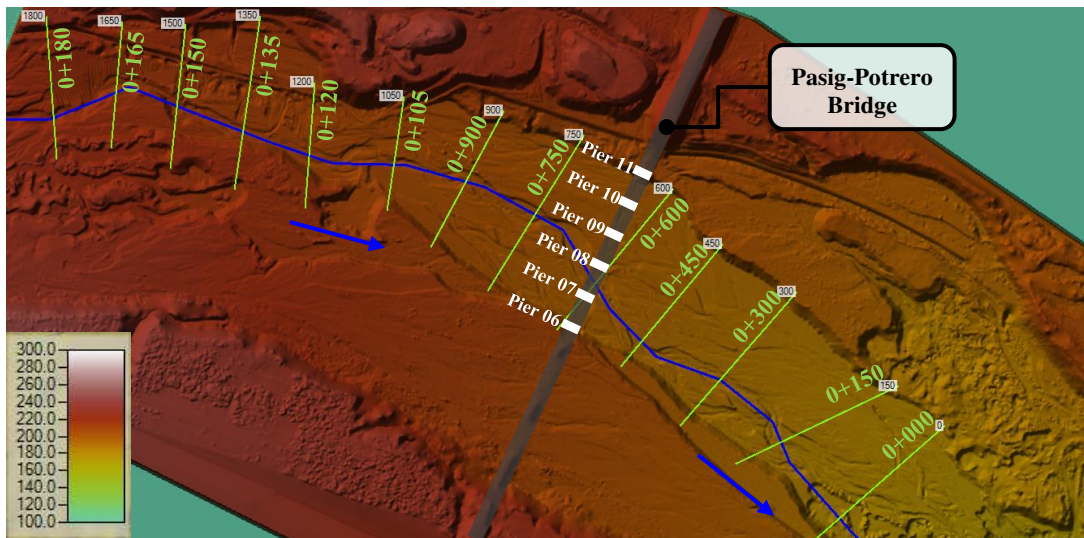

Figure 8. HEC-RAS Model of Pasig-Potrero River

Based on the results of the one-dimensional steady flow simulation, the flood levels (relative to the channel bed) show that the water within the river would not overtop the banks. The following table shows the discharge, flood level, and velocity at the bridge per return period.

Table 1. Hydraulic Model Results at the Bridge

Return Period (years)	Q (m³/s)	Flood Level (m)	Velocity (m/s)
25	234.24	1.88	2.36
50	265.68	1.95	2.38
100	297.22	1.99	2.47
100 (3-day)	780.93	2.56	3.02

For this study, bridge scour analysis was done within the HEC-RAS software. Since the hydraulic simulation considers one-dimensional steady flow, flow distribution was applied to consider the percentage of flow, flow area, wetted perimeter, conveyance, hydraulic depth, and average velocity for each flow element (left overbank, main channel, and right overbank). The contraction scour, or the movement of sediments due to change in cross-section (e.g. at the bridge) resulting in higher velocities, is computed using the Colorado State University formula, as follows:

$$y_s = 2.0 K_1 K_2 K_3 K_4 a^{0.65} y_1^{0.35} Fr_1^{0.43}$$

Where,

y_s	=	scour depth (m)
K_1	=	correction factor for pier nose shape (1.1 for square nose)
K_2	=	correction factor for angle of attack
K_3	=	correction factor for river bed condition
K_4	=	correction factor for bed material armoring
a	=	width of pier (m)
y_1	=	depth of flow at approach (m)
Fr_1	=	Froude number

The formula considers several factors such as the pier nose shape, angle of attack of the flow, the riverbed condition, the bed material, as well as the pier width and depth of the flow. The corresponding measurements and coefficients were inputted in the HEC-RAS software for the bridge scour model. For this study, the piers were assumed to have a square nose shape to represent the sheet piles.

The local pier and abutment scour were calculated using the Froehlich's Equation. The summary of the resulting scour depths for each return period is shown in Table 2. The resulting maximum local scour at the bridge piers, produced by the flows corresponding to the 3-day 100-yr probable maximum rainfall, are shown in Figure 9.

$$y_s = 2.27 K_1 K_2 L'^{0.43} y_a^{0.57} Fr_1^{0.61} + y_a$$

Where,

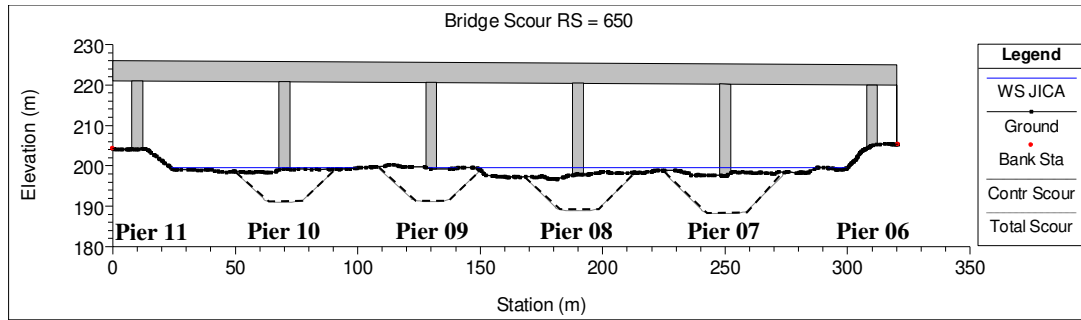|		
y_s	=	scour depth (m)
K_1	=	correction factor for abutment shape (0.55 for spill-through abutment)
K_2	=	correction factor for angle of attack
L'	=	length of abutment projected normal to flow (m)
y_a	=	average depth of flow at the approach section (m)
Fr	=	Froude number

Figure 9. Maximum Local Scour at the Bridge Piers

Table 2. Scour Depth at the Piers

Return Period (years)	Pier 11 (Abutment)	Pier 10	Pier 09	Pier 08	Pier 07	Pier 06 (Abutment)
25	0.94	4.60	2.28	6.51	6.77	1.42
50	1.21	5.58	5.03	7.32	7.60	1.50
100	1.43	5.93	5.56	7.49	7.76	1.54
100 (3-day)	4.60	8.26	8.15	9.15	9.38	2.09

Based on the hydraulic analysis, the maximum local scour from Piers 7 to 10 is 9.38 m on Pier 7. Taking this scour depth and adding the current exposed height, the required embedment depth was computed using the methodology provided in the USS Steel Sheet Piling Manual. In this design manual, the lateral earth pressures that act on the sheet pile wall were evaluated in order to determine the required embedment depth and compute the maximum bending moments and stresses on the sheet piles. The results were then used to select the appropriate sheet pile section, waling, and anchorage system. Based on the borehole data, the soil is composed of very dense sand beyond scour depth. The soil was evaluated to have a unit weight of 20 kN/m^3, cohesion of 0 kPa (for sand), and angle of friction of 34 degrees.

These parameters resulted to a required embedment of 4 meters. This value was multiplied by a Factor of Safety of 1.4 to get the total design embedment of 5.6 meters. Using the computed maximum moment and the allowable bending stress 260 MPa of steel, the resulting section modulus was 3602.6 x 10-6 m^3/m. Considering the required section modulus, the section to be used is SP-6L from the JFE Steel Sheet Piles. 50 mm diameter tie rods spaced at 1 meter shall also be utilized for the connection of piles on opposite sides. The table below presents the sheet pile design per pier.

Table 3. Sheet Pile Design Summary

Pier No.	Sheet Pile Section	Length (m)	Tie Rod Spacing (m)
06	JFE SP-3W	13.1	2.5
07	JFE SP-6L	18.9	1.0

Pier No.	Sheet Pile Section	Length (m)	Tie Rod Spacing (m)
08	JFE SP-6L	18.9	1.0
09	JFE SP-6L	18.9	1.0
10	JFE SP-6L	18.9	1.0
11	JFE SP-3W	13.1	2.5

4. GROUND SILL

Since lahar deposits could still be found upstream of the Pasig-Potrero River, it was assumed that sediments could still be transported to the Pasig-Potrero Bridge and accumulated by providing a ground sill. The ground sill was proposed to be constructed 50m downstream of the bridge and with a 5m height that would enable sediment accumulation to reach a level that is 2m above the pile caps. The channel bed should be backfilled up to the design elevation of 198m. Due to economic reasons, the ground sill was recommended to be constructed with gabions, with adequate anchorage to the banks and the bed.
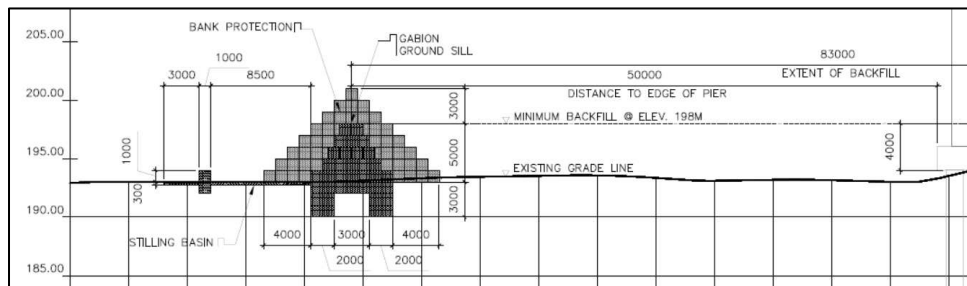

Figure 10. Proposed Ground Sill Scheme

An inline structure 50m downstream of the bridge was incorporated in the hydraulic model, and the upstream section was backfilled and assumed the same slope as the existing channel bed. The resulting maximum water surface elevation at the ground sill is 199.4m (Figure 11).
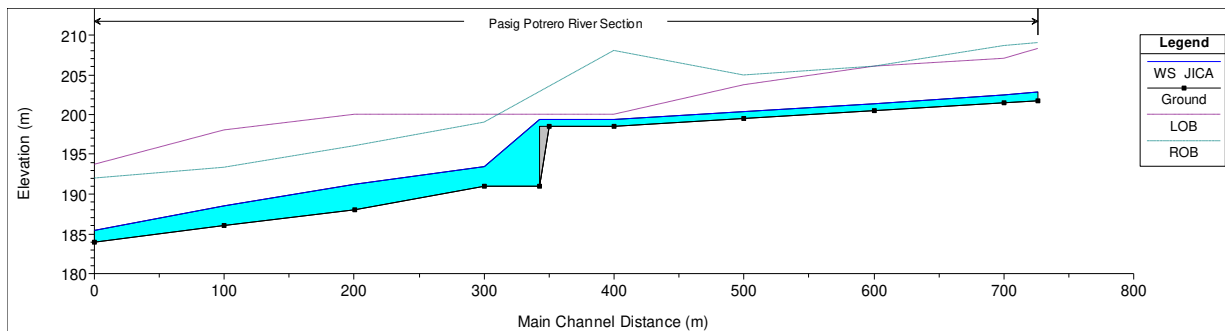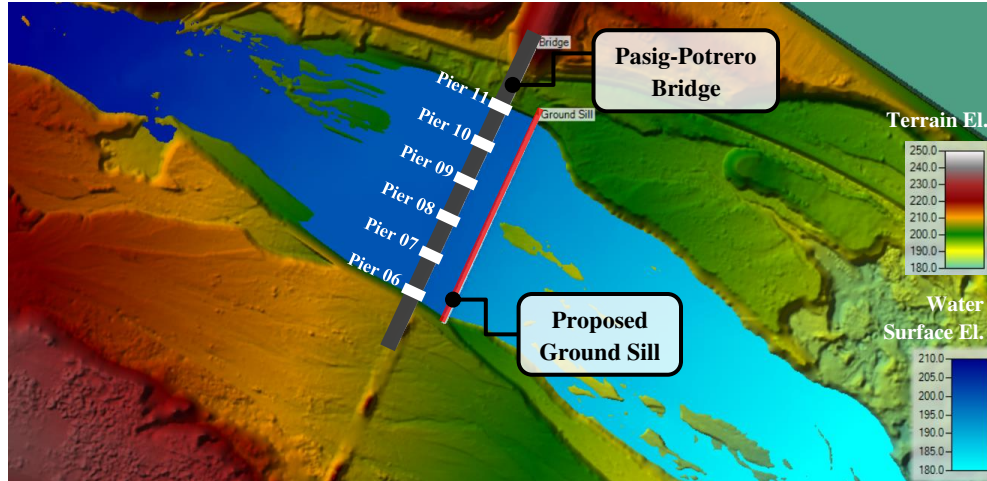

Figure 11. Flood Level Profile

Figure 12. Maximum Water Surface Elevation

To provide adequate allowance, gabion bank protection of up to 3m above the ground sill was therefore recommended. This is to prevent the bank from scouring and causing the ground sill to fail. Moreover, a stilling basin (also made of gabions) was recommended to be constructed immediately downstream of the ground sill. The stilling basin should be hinged to the ground sill in order to act as a launching apron. Launching aprons are non-rigid so that in the case of further scouring downstream, the apron would still be able to protect the ground sill structure from damage (Figure 13).
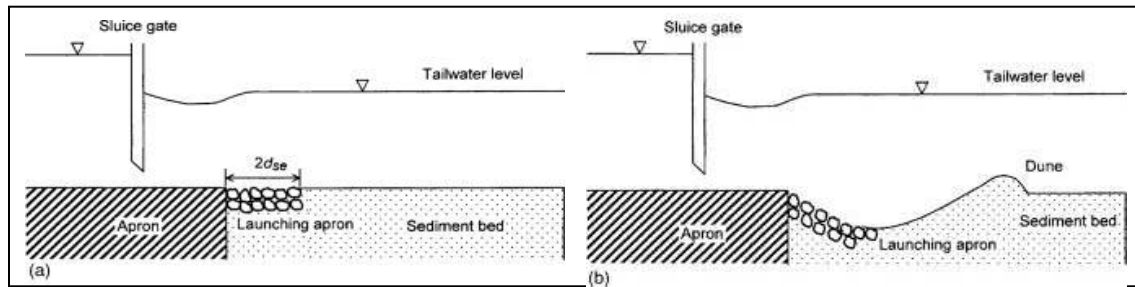

Figure 13. Launching Apron Scheme (Source: ASCE)

Due to the nature of the flow, the downstream of the proposed ground sill was identified to be prone to local scouring. The embedment depth of the ground sill shall therefore incorporate the computed scour depth. According to the DPWH Design Guidelines and Criteria and Standards for Water Engineering Projects, the scour depth at the downstream of the ground sill could be computed from the following equation:

$$d_s = (K_u H_t^{0.225} q^{0.54}) - d_m$$

Where,

d_s = local scour depth for a free overfall, measured from the streambed downstream of the drop (m)

q = discharge per unit width (m³/s/m)

H_t = total drop in head, measured from the upstream to downstream energy grade line (m)

d_m = tailwater depth (m)

K_u = 1.9

Based on the results of the hydraulic analysis for the 3-day rainfall flood, the total drop in head is equal to 5.87m. Applying the equation for scour depth, the maximum local scour depth immediately downstream of the ground sill is 2.52m. This could be mitigated by embedding the ground sill at a deeper elevation than the scour depth, or by constructing a launching apron which could both prevent scouring and dissipate energy coming from the head drop. The embedment of the ground sill was therefore set to 3m.
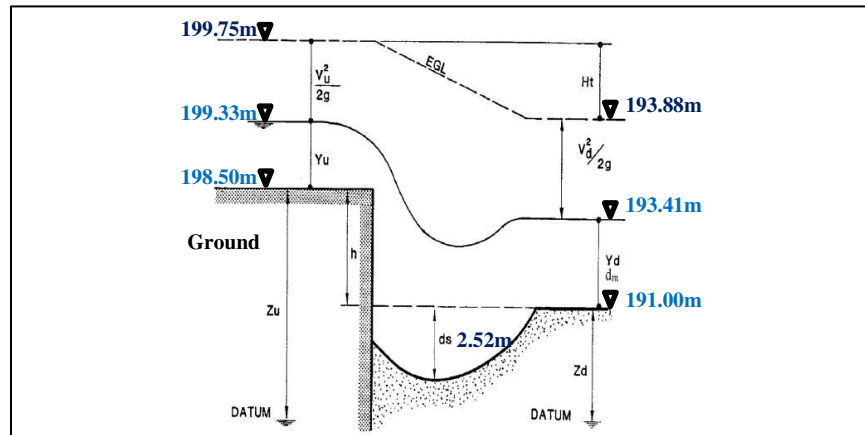

Figure 14. Downstream Erosion at Ground Sill

Based on the available geotechnical data and series of site inspections, the following geotechnical parameters were adopted in the slope stability analysis. Table 4 shows the geotechnical parameters used.

Table 4. Strength Parameters for Slope Stability Analysis of Ground Sill

Depth, m	Soil Type	Relative Condition / Consistency (N_{des})	Υ (kN/m³)	Geotechnical Parameters	
				c (kPa)	φ (⁰)
0.0 – 1.0	Sand	Very Dense	20	0	40
1.0 – 2.0	Sand	Medium Dense	18	0	35
2.0 – 3.0	Sand	Dense	19	0	39
>3.0	Sand	Very Dense	20	0	40
–	Backfill	-	18	5	30
–	Gabions	-	20	150	35

Slope Stability Analysis (SSA) was performed to determine the mitigation to be undertaken for the ground sill. A section was analyzed at the ground sill determining the Factors of Safety (FoS). For this analysis, two schemes were considered: (1) with backfill, and (2) without backfill. These schemes represent the different stages of construction for the ground sill. For static cases (case 2) and pseudo-static case (case 3), the required minimum FoS are 1.2 and 1.1, respectively. Based on the results of the SSA, adequate factors of safety are obtained for all four (4) scenarios (Table 5). The succeeding figures show the mathematical models of the scenarios showing the lowest factors of safety.

Table 5. Factors of Safety for the Ground Sill

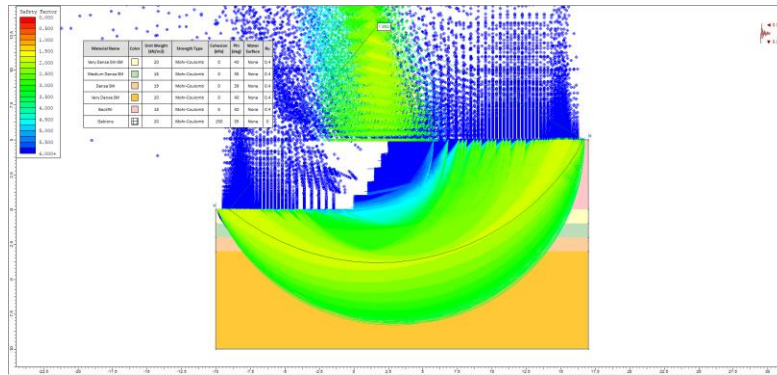| Scheme	Unsubmerged		Submerged	
	LC2	LC3	LC2	LC3
1	2.499	1.952	4.493	2.668
2	3.650	3.048	6.541	4.440
Remarks:	FS > 1.2	FS > 1.1	FS > 1.2	FS > 1.1

Figure 15. SSA Result for Scheme 1 - Case 3 Unsubmerged Condition (FoS = 1.952)
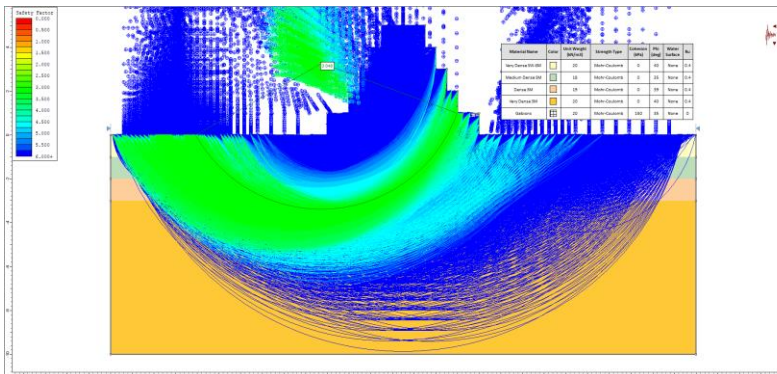

Figure 16. SSA Result for Scheme 2 - Case 3 Unsubmerged Condition (FoS = 3.048)

5.   CONCLUSION

SCTEX (Subic-Clark-Tarlac Expressway) is a major expressway located northwest of Manila. The imminent failure of the bridge would result in disruptions in transportation to the Subic, Clark, and Tarlac areas. Erosion of the channel bed and banks is continuous, and the resulting exposure of the piles and failure of the abutments could affect the structural integrity of the bridge. The immediate recommendation to protect the piers is to enclose them with backfill and sheet piles, which should be able to cover the pile caps. Monitoring of the sheet piles should be done if the channel bed further degrades.

The long-term protection of the piers, by restoring the design ground elevation of 203m using a ground sill shall still be prioritized. Lahar deposits found upstream of the Pasig-Potrero River could be transported and accumulated in the Pasig-Potrero Bridge using a ground sill. In case of slow accumulation of sediments, the channel bed could be backfilled up to a design elevation above the pile caps. Gabion bank protection should also be implemented to prevent the bank from scouring and causing the ground sill to fail. Moreover, a stilling basin hinged to the ground sill shall be constructed immediately downstream and act as a launching apron.

6.   RECOMMENDATIONS

The recommended pier protection scheme is only for the short-term protection of the piers, and structural design considerations such as the type of fill and depth of piles should be verified. As for the ground sill, the primary assumption for natural sediment accumulation is that lahar deposits are available to be transported from upstream of the river. A more detailed sediment transport analysis should be undertaken to determine the rate of accretion.

It should also be noted that the porous nature of the gabions was assumed to allow small flows through the ground sill. During strong flows, however, the ground sill would act as a weir. The hydrodynamic forces, based on the results of the hydraulic analysis (Figure 17), should also be determined to further verify the stability of the ground sill.
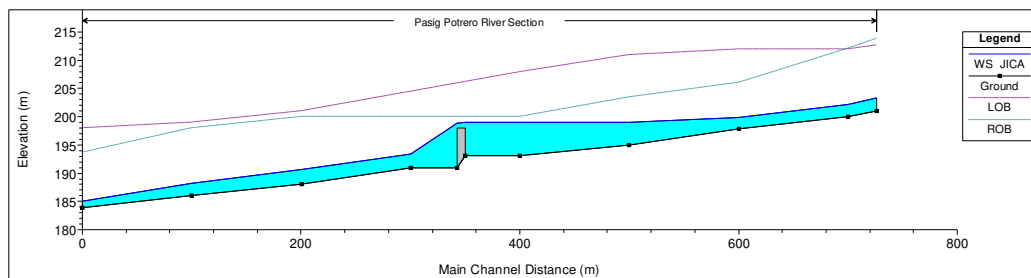

Figure 17. Ground Sill Modelled as a Weir

Aside from the structural measure described, non-structural measures are just as important in mitigating the erosion at the Pasig-Potrero Bridge. It is imperative that consistent monitoring of the mitigating measures and close coordination with concerned local government units and agencies are implemented. Regular monitoring of the pier protection scheme, at least semi-annually, should be done. Any change in the Pasig-Potrero River upstream may affect the conditions throughout the river (e.g. change in bed slope upstream could either lessen or aggravate bed degradation downstream) and should also be monitored. Human activities such as quarrying that could affect the rate of aggradation/degradation should also be addressed with concerned government agencies accordingly.

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